

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

Development Application for LYNDE & DICK
Name of Applicant
Address of site 918 BARNEMBOY ROAD, PALM BEACH NSW 2108

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

I, JORGE CABACO on behalf of SOILSROCK ENGINEERING Pty Ltd
(Insert Name) (Trading or Company Name)

on this the 31/08/2021 certify that I am a geotechnical engineer or engineering geologist or coastal engineer as defined by the Geotechnical Risk Management Policy for Pittwater - 2009 and I am authorised by the above organisation/company to issue this document and to certify that the organisation/company has a current professional indemnity policy of at least \$2million.

I:

Please mark appropriate box

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

Geotechnical Report Details:

Report Title: GEOTECHNICAL SITE INVESTIGATION REPORT
Report Date: 31/08/2021
Author: JORGE CABACO
Author's Company/Organisation: SOILSROCK ENGINEERING Pty Ltd

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

ARCHITECTURAL DRAWINGS BY TAIT GOODMAN ARCHITECTURE OFFICE
JOB NO: A072/BARNEMBOY RD/2108
GEOTECHNICAL SITE INVESTIGATION REPORT BY SOILSROCK ENGINEERING DATE 31/08/21

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

Signature Jorge Cabaco

Name JORGE CABACO

Chartered Professional Status... BENG, MENG, CENG, MCA

Membership No. 3789414 (ENGINEER AUSTRALIA)

Company... SOILSROCK ENGINEERING Pty Ltd

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

Development Application for LYNNE & DIKE
 Address of site 918 BARNES JONES ROAD, PALM BEACH TSW 2108
 Name of Applicant

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

Geotechnical Report Details:

Report Title: GEOTECHNICAL SITE INVESTIGATION REPORT
 Report Date: 31/08/21
 Author: JORGE CASARAO
 Author's Company/Organisation: SOILS ROCK ENGINEERING PTY LTD

Please mark appropriate box

- ☒ Comprehensive site mapping conducted 13/08/21
 (date)
- ☒ Mapping details presented on contoured site plan with geomorphic mapping to a minimum scale of 1:200 (as appropriate)
- ☒ Subsurface investigation required
☐ No Justification
☒ Yes Date conducted 13/08/2021
- ☒ Geotechnical model developed and reported as an inferred subsurface type-section
- ☒ Geotechnical hazards identified
☒ Above the site
☒ On the site
☒ Below the site
☒ Beside the site
- ☒ Geotechnical hazards described and reported
- ☒ Risk assessment conducted in accordance with the Geotechnical Risk Management Policy for Pittwater - 2009
☒ Consequence analysis
☐ Frequency analysis
- ☒ Risk calculation
- ☒ Risk assessment for property conducted in accordance with the Geotechnical Risk Management Policy for Pittwater - 2009
- ☒ Risk assessment for loss of life conducted in accordance with the Geotechnical Risk Management Policy for Pittwater - 2009
- ☒ Assessed risks have been compared to "Acceptable Risk Management" criteria as defined in the Geotechnical Risk Management Policy for Pittwater - 2009
- ☒ Opinion has been provided that the design can achieve the "Acceptable Risk Management" criteria provided that the specified conditions are achieved.
- ☒ Design Life Adopted:
☐ 100 years
☒ Other 50
 specify
- ☒ Geotechnical Conditions to be applied to all four phases as described in the Geotechnical Risk Management Policy for Pittwater - 2009 have been specified
- ☐ Additional action to remove risk where reasonable and practical have been identified and included in the report.
- ☐ Risk assessment within Bushfire Asset Protection Zone.

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

Signature Jorge Casarao
 Name JORGE CASARAO
 Chartered Professional Status SENG, MEMO, CENG, MER
 Membership No. 3789414 (ENGINEERS AUSTRALIA)
 Company SOILS ROCK ENGINEERING PTY LTD

**GEOTECHNICAL SITE INVESTIGATION REPORT
FOR
PROPOSED NEW DWELLING
AT
918 BARRENJOEY ROAD, PALM BEACH, NSW 2108**



Report Prepared for: LYNDE & DICK

Project No: SRE/844/PB/21

Date: 31/08/2021

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Distribution and Revision Register

Document details

- **Project number:** 844
- **Document number:** SRE/844/PB/21
- **Document title:** Geotechnical Site Investigation Report for Proposed New Dwelling
- **Site address:** 918 Barrenjoey Road, Palm Beach, NSW 2108
- **Report prepared for:** LYNDE & DICK

Document status and review

Revision	Prepared by	Reviewed by	Approved by	Date issued
0	SK	JC	JC	1/09/2021

Distribution of copies

Revision	Electronic	Paper	Issued to
0	1	0	Matt Ross Goodman
0	1	0	Edwin Hang Jiang

The undersigned, on behalf of Soilsrock Engineering Pty Ltd, confirm that this document and all attached documents, drawings, and geotechnical results have been checked and reviewed for errors, omissions, and inaccuracies.

For and on behalf of

Soilsrock Engineering Pty Ltd



Jorge Cabaco

BEng MEng MIEAust CPEng NER RPEQ VBA

Principal Geotechnical Engineer

ENGINEERS AUSTRALIA

Chartered Engineer INER - National Engineers Registration No. 3789414

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APPENDIX F – PRACTICE NOTE GUIDELINES FOR LANDSLIDE RISK MANAGEMENT AGS 2007 AGS – APPENDIX G – SOME GUIDELINES FOR HILLSIDE CONSTRUCTION INTRODUCTION

1. INTRODUCTION

This report presents and interprets the result of a geotechnical investigation assessment carried out by Soilsrock Engineering Pty Ltd (SOILSROCK) of the existing site at 918 Barrenjoy Road, Palm Beach, NSW 2108. The investigation was commissioned by Mr. Matt Ross Goodman and Mr. Edwin Hang Jiang, who are the architects and the representatives of the property. SOILSROCK conducted the work in general accordance as per email proposal on 2nd August 2021 and acceptance on the next day.

This assessment report comprised a detailed geotechnical inspection of the existing site and is based on the following documents supplied by the client on the email of 2nd August 2021:

- Survey drawing prepared by H & S LAND SURVEYORS PTY LTD, Project No; 21071, " SURVEY PLAN SHOWING DETAIL & LEVELS OVER LOT 16 IN DP 650061 NO. 918 BARRENJOY ROAD, PALM BEACHES NSW 2108, Rev A, Dated on 27-05-2021
- Architectural Sketch Design Drawings by MATT GOODMAN ARCHITECTURE OFFICE, "918 BARRENJOEY ROAD", Job No. A072/BARRENJOEY RD/2108, Date as provided:
 - "SITE SURVEY", DWG No. TP001.
 - "EXISTING & DEMOLITION SITE PLAN", DWG No. TP003.
 - "PROPOSED SITE & ROOF PLAN", DWG No. TP004.
 - "GARAGE FLOOR PLAN", DWG No. TP005.
 - "LOWER GROUND FLOOR PLAN", DWG No. TP006.
 - "GROUNF FLOOR PLAN", DWG No. TP007.
 - "FIRST FLOOR PLAN", DWG No. TP008.
 - "DEMOLITION ELEVATIONS", DWG No. TP009.
 - "PROPOSED ELEVATION – NORTH", DWG No. TP010.
 - "PROPOSED ELEVATION – EAST", DWG No. TP011.
 - "PROPOSED ELEVATION – SOUTH", DWG No. TP0012.
 - "PROPOSED ELEVATION – WEST", DWG No. TP013.
 - "PROPOSED ELEVATION – GARAGE", DWG No. TP014.
 - "PROPOSED ELEVATION – POOL", DWG No. TP015.
 - "PROPOSED SECTION – SHEET1", DWG No. TP016.
 - "PROPOSED SECTION – SHEET2", DWG No. TP017.
 - "PROPOSED SECTION – SHEET3", DWG No. TP018.
 - "SHADOW DIAGRAMS", DWG No. TP025.

The purpose of this investigation was to assess the existing subsurface ground conditions and risks associated with the existing slope versus new development construction and to provide

geotechnical recommendations and advice on excavation conditions, retaining walls design options, foundations design options and landslide risk assessment.

The following sections describe the proposed development, scope of works and factual results of this site investigation. Comments and recommendations on excavation and foundations conditions, including landslip risk assessment for the proposed dwelling is given in the last part of this report.

2. PROPOSED DEVELOPMENT

Based on the architectural drawings provided by the client, it is understood that a new dwelling is proposed to be constructed, after the demolition of the existing dwelling. Vehicular access can be made via the common concrete driveway for 916 and 918 Barrenjoey Road, Palm Beach, NSW 2108 located in front of 916 Barrenjoey Road property, at the South side of the property.

According to the garage floor plan, it will accommodate a two-space garage, a walkway space a store area and a carport. On the lower ground floor plan, it accommodates two bedrooms, a common rumpus room space for both the rooms, a powder room, a bathroom, a Lenin room space, stairs, and an inclinor stop the west side of the property which connects the garage floor to the ground floor level. On the ground floor plan, it accommodated a dining space, fireplace, living room, kitchen, pantry space, casual dining space, laundry, powder room, a balcony, and a BBQ space. On the first floor plan it accommodated a rumpus space, a bathroom, a study space, a main bedroom and stairs which connects the ground floor and the first floor. The proposed building is of a flat roof. Details of the proposed developments are shown on the architectural drawings provided by MATT GOODMAN ARCHITECTURE OFFICE referred above.

3. SCOPE OF WORKS

The field work for investigation was carried on the 13th August 2021 and consisted of the following:

- Conduct Dial Before You Dig and electronic scans for buried services.
- Conduct an OH&S and walkover survey to assess local topography, geology, hydrology, and existing site conditions, including exposed soil/rock conditions, vegetation, and surface drainage.
- Conduct a geotechnical inspection of the site area and adjacent land.
- 6 x Dynamic Cone Penetrometer tests (DCP1 to DCP6) to maximum depth of 2.90m were carried out by using a 9 kg Dynamic Cone Penetrometer specialised steel cone device. The testing followed the procedure as per AS 1289-1997, method 6.3.2.
- Photographic record of the site conditions.

The field work was conducted in presence of a registered professional senior geotechnical engineer, a geotechnical engineer, and an engineering assistant from Soilsrock office, who observed visually the existing geotechnical conditions and recorded the DCP in-situ test results.

The **Appendix A** defines and explains the logging terms and symbols used. The **Appendix B** show the plan of the Site Photos and the DCP test locations and site photographs of the area are attached to this report in the **Appendix D**.

4. RESULTS AND ANALISYS OF THE INVESTIGATION

4.1 Site Location and Description

The subjected site is located at 918 Barrenjoey Road, Palm Beach, NSW 2108, which belongs to the Northern Beaches Council and is legally registered as Lot 16, DP 650061 within E4 – Environmental Living land-use zoning. The site has a rectangular shape which covers a plan area of approximately 851.2m². The site topography is a steep slope with over 25 degrees, slopping down from the North to the South. The surrounding area is surrounding mainly for residential purposes. Site and Dynamic Cone Penetrometer testing (DCP) locations are shown in **Appendix B**, and site photographs of the area are attached to this report in **Appendix D**.

4.2 Regional Geology

From the analysis of Geology of the Sydney 1:100 000 sheet 9130, it is indicated that the site is located within the geology units of Reference “**Rnn**”, age of Triassic, described as “interbedded laminite, shale, and quartz to lithic-quartz sandstone. The site belongs to of Newport Formation and Garie Formation of Narrabeen Group.

A reproduction of the geological map is shown in **Figure 1** and is based on a portion of the Sydney 1:100 000 Geological Series map 9130 (interactive resource provided by the Geological Survey of NSW), which shows the site geological condition.

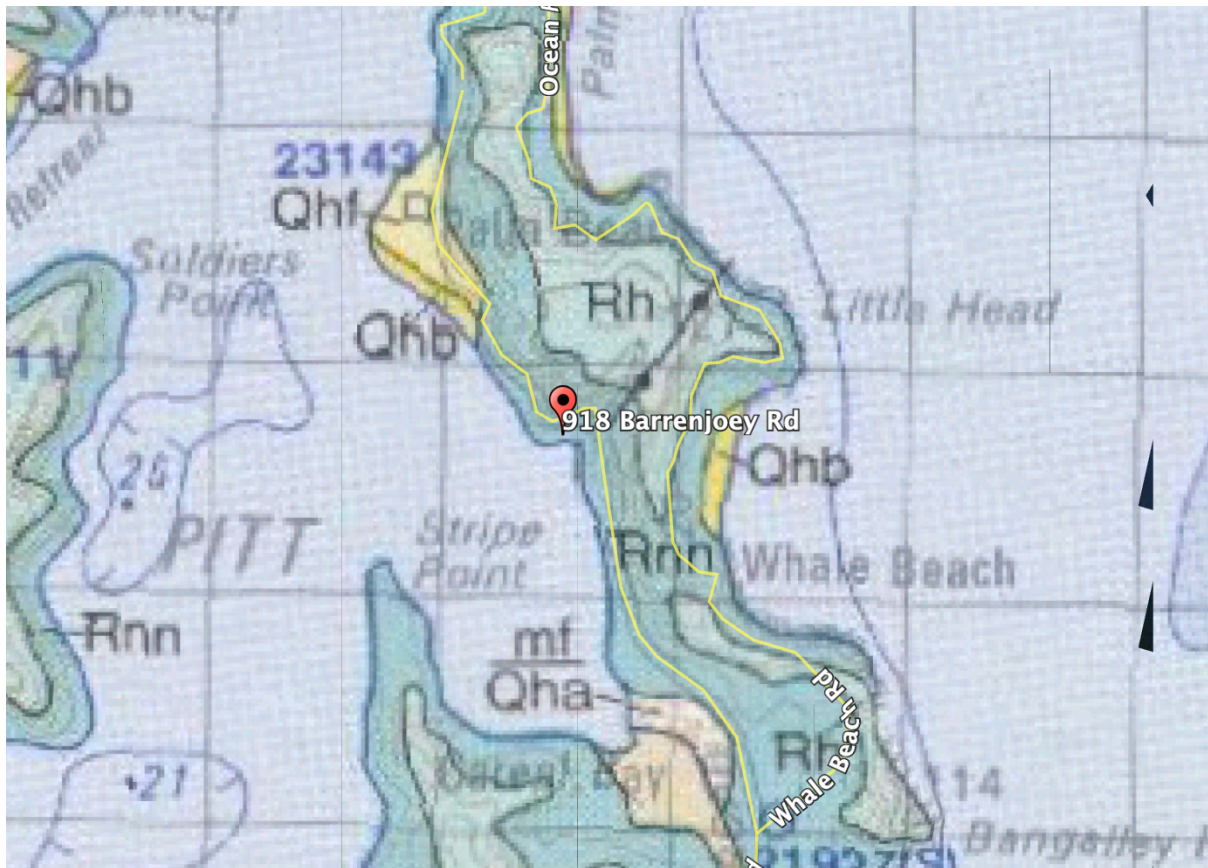


Figure 1 – Portion of the Sydney 1:100,000 Geological Series map 9130. Site area location is highlighted in a red/black sign.

4.3 Subsurface Investigation

Six Dynamic Cone Penetrometer (DCP) tests were carried out to complement the investigation of subsurface ground conditions. The following **Table 1** summarised the in-situ DCP test results and **Table 2** describes generically the principal strata sequentially observed and interpreted by the test results carried out on site.

Table 1 - Dynamic Cone Penetrometer tests results – DCP1 to DCP6.

Depth (m)	DCP1 (Blows/ 300mm)	DCP2 (Blows/ 300mm)	DCP3 (Blows/ 300mm)	DCP4 (Blows/ 300mm)	DCP5 (Blows/ 300mm)	DCP6 (Blows/ 300mm)
0.00 – 0.30	4	1	3	2	6	2
0.30 – 0.60	10	2	1	2	11	8 Bouncing @ 0.55m Probably on top of rock
0.60 – 0.90	31	11	5	4	10	-
0.90 – 1.20	30 Bouncing @ 1.0m Probably on top of rock	33	14	4	5	
1.20 – 1.50	-	17	25	30	10	
1.50 – 1.80		17	34	Refusal @ 1.8m	11	
1.80 – 2.10		19	34	-	33	
2.10 – 2.40		36 Bouncing @ 2.4m Probably on top of rock	22		Refusal @ 2.25m	
2.40 – 2.70		-	32		-	
2.70 – 3.00			Refusal @ 2.9m			

Equipment & Procedure Notes:

Equipment used: 9kg hammer, 510mm drop distance, conical tip: Standard used: AS1289.6.3.2 – 1997; the total number of blows are considered for 300mm penetration steps.

DCP notes:

- 60 blows within 300mm soil interval defined as “Refusal”, no further penetration and “Solid” ringing sound from slide hammer, which may indicate reaching into “Very dense” sand layer or “Hard Clay” or on top of bedrock.
- All DCP tests above which were at refusal depths may probably still be on top of hard clay.
- “Bouncing” indicates reached top of rock or in some cases can be due to presence of a hard obstacle like steel, rubble, flouters, boulders, cobbles, or other hard materials.

Table 2 - Geotechnical subsurface interpretation by in-situ DCP results – DCP1 to DCP6.

Depth (m)	DCP1 (Blows/ 300mm)	DCP2 (Blows/ 300mm)	DCP3 (Blows/ 300mm)	DCP4 (Blows/ 300mm)	DCP5 (Blows/ 300mm)	DCP6 (Blows/ 300mm)
0.00 – 0.30	Loose Sand				Loose Sand	Very Loose Sand
0.30 – 0.60	Medium Dense Sand	Very Loose Sand	Very Loose Sand	Very Loose Sand		Very Dense Sand Probably on top of rock
0.60 – 0.90	Dense Sand		Loose Sand			
0.90 – 1.20	Very Dense Sand Probably on top of rock			Loose Sand	Medium Dense Sand	
1.20 – 1.50				Dense Sand		
1.50 – 1.80				Very Dense Sand		
1.80 – 2.10					Dense Sand	
2.10 – 2.40	-	Very Dense Sand Probably on top of rock			Very Dense Sand	
2.40 – 2.70			Dense Sand			
2.70 – 3.00			Very Dense Sand			

Note: No samples were provided by the DCP test, thus the geotechnical interpretation above is based only on the observation carried through the soil traces left attached to the rods and tip; This subsurface interpretation is based on DCP result obtained in **Table 1** and subsequent professional engineering judgement. Hence, the interpretation is only indicative, and some soils characteristics can be difficult to identify properly without samples; “Probably on top of rock” indicates reached top of rock or in some cases can be due to the presence of hard obstacles such as steel, rubble, flouters, boulders, cobbles or any other hard obstacles.

The above DCP’s Tests locations are shown in the **Appendix B**. The **Table 3** below assesses the strength of the relevant materials crossed by the DCP tests, according to in-situ test results, soil classification, visual interpretation, and extrapolation.

The geotechnical parameters interpretation and extrapolation is based and limited to DCP tests carried on site, which are only indicative for design proposes.

For detailed description of the subsurface conditions, explanation sheets about geotechnical parameters are presented in **Appendix A**.

Table 3 - Allowable Bearing Pressure and Strength Interpreted and Extrapolated by in-situ tests.

Depth Range (m)	Material Conditions	Extrapolated Allowable Bearing Pressure (kPa)	Strength (Friction Angle = ϕ)
Based on DCP1 Test Results			
0.00 - 0.30	Loose Sand	50	25°
0.30 - 0.60	Medium Dense Sand	100	30°
0.60 - 0.90	Dense Sand	300	35°
0.90 - 1.00	Very Dense Sand	500	40°
Based on DCP2 Test Results			
0.00 - 0.60	Very Loose Sand	NR	NR
0.60 - 2.10	Medium Dense Sand	100	30°
2.10 - 2.40	Very Dense Sand	500	40°
Based on DCP3 Test Results			
0.00 - 0.60	Very Loose Sand	NR	NR
0.60 - 0.90	Loose Sand	50	25°
0.90 - 2.40	Medium Dense Sand	100	30°
2.40 - 2.70	Dense Sand	300	35°
2.70 - 2.90	Very Dense Sand	500	40°
Based on DCP4 Test Results			
0.00 - 0.60	Very Loose Sand	NR	NR
0.60 - 1.20	Loose Sand	50	25°
1.20 - 1.50	Dense Sand	300	35°
1.50 - 1.80	Very Dense Sand	500	40°
Based on DCP5 Test Results			
0.00 - 0.30	Loose Sand	50	25°
0.30 - 1.80	Medium Dense Sand	100	30°
1.80 - 2.10	Dense Sand	300	35°
2.10 - 2.25	Very Dense Sand	500	40°
Based on DCP6 Test Results			
0.00 - 0.30	Very Loose Sand	NR	NR
0.30 - 0.55	Very Dense Sand	500	40°

Notes:

- The geotechnical parameters interpretation and extrapolation is based and limited to the DCP test carried on site, which are only indicative for design proposes.
- The depth ranges of geological units as shown in the table are average thickness based on DCP test results obtained. It is understood that the subsurface conditions can vary from places to places.
- NR – Not Recommended.

As indicated within the table above, some of the DCP's tests recorded "bouncing", the DCP rods were bouncing back by hammer striking on top of the rods at the end of the tests which could indicate that it has probably reached the top of the rock.

The regional geology indicates that the site is underlain by sand and silt which will probably direct to sandstone or shale, therefore the following **Table 4** indicates the interpreted and inferred geotechnical parameters for sandstone/shale rock if encountered during excavations for drilling and piling construction. The following rock parameters are given for the lowest rock quality, regarding the hand methods by DCP tests are not able to investigate the rock in deep. In addition, the below geotechnical parameters should not be used if it is confirmed the presence of rock boulders and floaters within the site, further geotechnical inspections and testing must be undertaken to confirm properly the geotechnical parameters for rock at the specific locations.

Table 4 – Recommended Geotechnical Parameters for Rock

Foundation Stratum	Allowable End Bearing Pressure (kPa)	Ultimate End Bearing Pressure (kPa)	Ultimate Shaft Adhesion (kPa)	Typical Elastic Modulus (MPa)
Class V	700	3,000	50	50

Notes:

- Rock Classification and bearing pressures based on P.J.N Pells "Substance and Mass Properties for The Design of Engineering Structures in The Hawkesbury Sandstone" AGM Vol No. 39 September 2004
- Ultimate end bearing pressures values occur at large settlements (>5% of minimum footing dimensions)
- Ultimate shaft adhesion values to depend on clean socket of roughness category R2 or better. Values may have to be reduced because of smear.
- Shaft adhesion applicable to the design of CFA or bored piles, uncased over the rock socket length, where adequate sidewall cleanliness and roughness are achieved.

To clarify the rock quality and to determine if rock boulder and floaters are present within the site, to assist retaining walls and foundations design, it is recommended that an additional geotechnical investigation by rock core drilling to core the rock and permit carry strength tests such as "IS50 - Point Load Tests" should be carried out, considering minimum 2 boreholes to 5-6m depth each prior the excavation works commence. The rock coring can be carried by a mini drill rig or hand portable drill mast depending on the type of accessibility to the site can be created.

4.4 Groundwater

According to the Geotechnical investigation groundwater was not recorded on the DCP tests rods when extracted from the ground. However, groundwater can be investigated properly by further geo-hydrological assessment using a proper drilling and standpipe installation to monitor groundwater if required.

5. COMMENTS AND RECOMMENDATIONS

5.1 Landslip Risk Assessment

During the site inspection and in-situ testing, it was observed signs of cracking and some soil erosion within the landscape area located at the northern and southern part of the site.

The site is located within a “Geotechnical Hazards Area H1”, accordingly with the Pittwater Geotechnical Hazard Map from Pittwater LEP Local Environment Plan 2014.

A reproduction of the Pittwater Geotechnical Hazard Map is shown in **Figure 2** and is based on a portion of the Geotechnical Hazard Mapping LGA 2007 – GHD Longmaz from the Old Pittwater Council, which shows the site geological condition as follow:

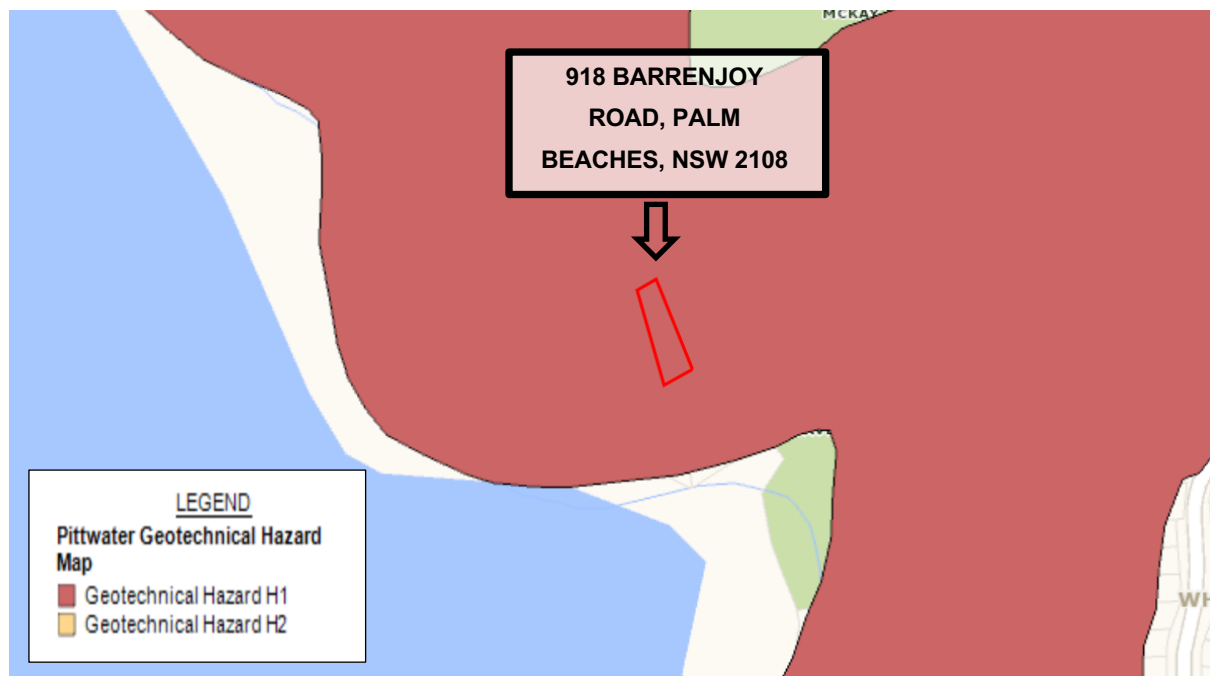


Figure 1 – Portion of the Pittwater Geotechnical Hazard map. Site area is highlighted in Red.

Nevertheless, some hazards have been identified and assessed for risk to property and life using the general methodology outline by the Australian Geomechanics Society (Landslide

Risk Management AGS Subcommittee 2007), the risk assessment is outlined on the following **Table 4**.

Table 4 – Geotechnical Hazards Summary Risk Analyses

HAZARDS	*Qualitative Measures of likelihood	*Qualitative Measures of Consequences to Property	*Risk to Property	*Risks To Life	*Level Risk Implications
Soil creek cause cracking on residential dwelling due to big rain events	Unlikely - (annual probability $P_{(H)} = 10^{-4}$)	Medium (20%)	Low (6.3×10^{-5})	1.4×10^{-7} /annum	**Risk Acceptable
Soil erosion weakens tree roots and causes trees falling down and potential landslides.	Unlikely - (annual probability $P_{(H)} = 10^{-4}$)	Minor (5%)	Low (6.3×10^{-5})	1.4×10^{-7} /annum	**Risk Acceptable
Soil erosion exposes rock boulders and outcrops and causes potential rockfall.	Rare – (annual probability $P_{(H)} = 10^{-5}$)	Major (60%)	Low (6.3×10^{-6})	2.7×10^{-7} /annum	** Risk Acceptable
Soil erosion causes land surface on or above the property failing and impacting on the proposed subdivision	Unlikely – (annual probability $P_{(H)} = 10^{-4}$)	Medium (20%)	Low (6.3×10^{-5})	1.4×10^{-7} /annum	**Risk Acceptable
"Rapid failure of open excavation during piling works for lower ground and garage levels construction	Rare – (annual probability $P_{(H)} = 10^{-5}$)	Major (60%)	Low (6.3×10^{-6})	2.7×10^{-7} /annum	**Risk Acceptable
Slow failure of new retaining walls	Rare – (annual probability $P_{(H)} = 10^{-5}$)	Medium (20%)	Low (6.3×10^{-6})	1.4×10^{-8} /annum	**Risk Acceptable

Note: *Refer to Australian Geo-Mechanics Vol. 42 No. 1 March 2007, for full explanation of terms above; **Level of Risk Acceptable: AGS Suggested Tolerable loss of life individual risk = 10^{-4} /annum for existing slope/ existing development (**Appendix E**)

Following the above, it is considered that the current site meets “Acceptable Risk Management” criteria with respect to both property and life under current and foreseeable conditions. As indicated by the DCP tests results, it is also noted the soils consists of very

loose to very dense sand present on the proposed development area to be at maximum depth of 2.9m.

It is important to clarify that the present risk assessment for landslip only takes in consideration of local slope stability including the existing property and adjoining structures. For global landslip risk analyses which includes a much larger area including many residences from the top of the slope to down slope and sides, will require a separate global risk assessment landslide analyses and report which should be undertaken by the official governmental entities if required.

Following the above, after the demolition of the existing building and prior to start any excavations to construct the proposed new dwelling, a site geotechnical inspection by a professional geotechnical engineer must be undertaken to observe the slope and clarify the need to install a permanent or temporary shoring wall. Batters slope excavation could be considered if there is enough space to excavate by batters with maximum inclination of 1V:1.5H as the excavation progresses, to stabilise and retain the back of the property and neighbouring boundaries of the property, to permit to excavate safely and construct the lower ground floor and garage floor levels of the proposed development. If vertical excavations are required, temporary or permanent retaining walls are required. In addition, as mentioned above rock coring is recommended to assess rock quality and strength and check if boulders or floaters are present within the site.

Providing the batters slope excavation are not over 1V:15H and or shoring walls are constructed accordingly if required it is considered that the proposed development will meet “Acceptable Risk Management” criteria with respect to both property and life upon appropriate application of geotechnical recommendations in this section, proper engineering design and construction methodology, and adequate on-site supervision and assessment by a professional chartered registered geotechnical engineer.

To maintain a good hillside construction practice, the following are recommended for the proposed development (refer to **Appendix F**):

- Appropriate surface water drainage must be installed to avoid excessive water infiltration through the ground.
- Appropriate roof water piped and connected properly to the stormwater street systems to avoid excess water infiltration through the ground.
- Piles and footings must be socket into competent rock to allow for landslide risk.

- Cutting and filling should be minimized to reduce site disturbance within a landslide risk area.
- Sewage effluent pumped out or connected to sewer tanks shall be adequately founded and watertight.

5.2 Excavation Conditions & Seepage Conditions

The supplied architectural drawing plans indicate that from lower ground to the first-floor levels, could require excavation depths range from 2-3m. Based on the in situ testing the cuts are expected to be carried out through the very loose sands to dense sands material to maximum depths of 2.9m excavation, further down it is expected rock excavation. At the bottom slope level for the garage construction, it is expected that excavation depth could reach 3.7m. In addition, excavation for deep footings or piles could be required to minimum 1.0m socket into solid rock regarding the landslide risk and good engineering practice on hillside construction (refer to **Appendix F**).

Excavation of the soil profile and weak rock can be completed using conventional earthworks equipment such excavators equipped with bucket to cut the sandy soils and extremely weathered, and very low strength rock will be suitable. If better strength and quality rock above medium strength rock are encountered at shallow depths, regarding the proximity of the neighbour buildings, it is recommended to cut the rock by sawing methods, this will reduce considerably the vibration and minimise noise levels to the surrounding building structures. Therefore, there will be no adverse impact on the surrounding properties and infrastructures due to vibration created by the excavation saw cutting method mentioned above when rock is intersected.

It is recommended that the demolition of the existing structures, excavation and construction techniques be adopted without causing more than 5mm/sec vibrations limit (Peak Particle Velocity) to the existing neighbouring buildings. If the existing neighbours' houses are constructed in weak conditions vibration limits could be necessary to be reduced to 3mm/sec vibrations limit (Peak Particle Velocity). If necessary, a vibration monitoring plan should be implemented to control vibration levels. Vibration monitoring plan must be carefully planned by the builder and will depending on the rock cutting methods by saw cutting, small rippers, small hammers to detach rock, and small size of excavators employed prior start demolition and excavation works.

Dilapidation survey report for the neighbouring residential building could also be required to record the current building conditions prior to the commencement of site works including

excavation and any necessary shoring/retaining walls on site. These will document any defects within the building(s) so that any claims for damage due to vibration can be properly assessed.

Regarding the site is on the slope it is not expected groundwater high flows. However, rainwater/groundwater seepages could be retained within the rock fractures which by excavation could be released, which could require an intermittent dewatering during excavation and construction works.

A Waste Classification should be carried for all the excavated materials to be disposed in accordance with NSW Environment Protection Authority (EPA) Waste Classification Guidelines Nov 2014, and under the Protection of the Environment Operations Act 1997 (POEO Act). Environmental sampling and chemical laboratory testing will need to be carried out to classify the spoil resulted from the excavation prior to disposal. This includes filling and excavated natural materials (GSW/VENM/ENM), if it is intended to be removed from the site. The type and extent of testing undertaken will depend on the final use or destination of the spoil, and requirements of the site.

5.3 Excavation Support & Retentions Systems.

As mentioned above, if excavation by batters with maximum inclination of 1V:1.5H cannot be considered regarding space limitations, ground support or retaining structures are required.

For the excavation to construct the garage and above floor levels, shotcrete wall combined with ground anchors are recommended. Excavation drops must be not more than 1m deep in soils to ensure stability of the excavation during the drilling to install ground anchors and preparation of the steel mesh for further shotcrete. If weak rock is encountered a drop of 1.5m deep could be considered subject to an inspection of a chartered registered professional geotechnical engineer. If good quality of rock of over 3,500kPa allowable bearing pressure is encountered vertical excavation without support can be considered again subject to a geotechnical inspection confirmation.

If anchors are not allowed for ground support/retaining wall solutions, piling shoring walls by concrete bored soldier piles combined with shotcrete or timber sleepers (providing steel UC beams are installed within the piles to allow installation of timber sleepers) in cantilever are required.

5.4 Foundations – Footings & Piles

The foundations conditions across the footprint of the new building development are expected to intersect very loose to very dense sand materials to 2.90m deep underlying by probably rock materials.

Further to the results of the investigation and architectural drawings described above, it is recommended installing deep footings or footings supported by bored piles socket minimum 1.0m into solid good quality rock. Carefully must be taken to ensure that the foundations are not installed within and above rock boulders and/or floaters.

It is also recommended that the footings to be founded in an appropriated ground allowable bearing pressures determined by the footing design, depending on the loads considered, size and type of footings. Either way, the foundations of the entire building must be installed and socket to ensure stability of the footing/pile in competent solid rock materials (loose or debris materials must be removed prior to footing construction) to prevent against landslide regarding the property is on the high Geotechnical Hazard Zone 1.

However, the founding depths must be adjusted and confirmed by the structural loads and foundations type required for the project. During the excavation to install the footings/piles, it is essential the ground foundations materials to be inspected and approved by a qualified professional registered geotechnical engineer to ensure ground materials and bearing pressures are as expected.

Once the structural loads and footings designs have been confirmed, settlement analysis should be carried out to confirm the suitability of the foundation solution adopted.

All piles/footings excavation base should be dewatered, cleaned, and be free of any loose material prior to pouring. Time between footing/piles excavation and concrete pour must be kept to the minimum, and delays are anticipated, it is recommended that the base of the footings be protected by a blinding layer of concrete with minimum strength of 25Mpa immediately after excavation to reduce any potential “loosening” effects.

All foundations should be designed and constructed in accordance with AS 2870 – 2011 – “Residential Slabs and Footing”.

5.5 Final Comments and Conclusions

Further to the above, additional geotechnical input is required and summarized as follow:

- Carry additional geotechnical investigation after demolition of the existing building by carrying 2 to 3 boreholes to 6m deep each by rock core drilling to determine rock quality at the vertical deep excavation locations to reach the first and lower ground floors and garage levels.
- Develop and concept a ground support/retaining wall design solution for the excavation of the lower and garage levels.
- Geotechnical monitoring program to control and ensure low noise and vibrations to neighbour residence buildings prior start and during the demolition and excavation works if required.
- Dilapidation reports to adjoining residential buildings regarding the demolition and excavation works.
- Geotechnical site inspections during excavation works, shoring wall, footings, and piles to confirm soil and rock bearing capacities.
- DCP tests to confirm soils foundations bearing pressures for footings, slabs and pavements if required.

Further to the results of the investigations and geotechnical recommendations above, providing the works are carried accordingly with this report, and good engineering and building construction practice on hillside construction is maintained the proposed development is suitable for the site.

The geotechnical bearing pressures recommended on this report are based on the testing locations and on the in-situ soils investigated in deep. However, the geotechnical bearing capacities could vary across the site outside of those locations, the founding depth for foundations to be constructed could also vary as a result. Therefore, it is recommended that during the excavation and foundation's installation, specialised personnel such as an experienced professional registered geotechnical engineer should inspect and approve the excavation works and founding levels.

6. LIMITATIONS

The site geotechnical investigation undertaken for the present report is an interpretation and estimation of the characteristics of the soil and or rock of subsurface conditions encountered during the test locations points investigated. No matter how comprehensive the investigation is, site ground conditions in other test locations investigated can differ and geological/geotechnical conditions can be unpredictable or can reveal unforeseen conditions.

The present report analyses form an engineering model interpretation and opinion of the actual subsurface conditions of the locations points where the tests were carried. The selected in-situ tests results are indicative of actual conditions encountered on the location points investigated. Recommendations are given based on the data testing results and visual interpretation carried by professional geotechnical and geological engineers from this office. Interpretation of the present report by others may differ from the interpretation given, there is the risk the report may be misinterpreted and Soilsrock cannot be held responsible for this.

Geotechnical reports rely on factual interpreted and judgement of information based on professional visual interpretation of soils and rock samples, in situ and sampling tests, which can have some uncertainty due to unexpected natural and normal changing ground conditions. Soilsrock Engineering accepts no responsibility if different unexpected ground conditions occur in locations where the investigations were not carried out.

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APPENDIX A

GEOTECHNICAL EXPLANATORY NOTES

APPENDIX A – GEOTECHNICAL EXPLANATORY NOTES

The following geotechnical notes are provided, to give a better understanding of the description and classification methods and field procedures used for the interpretation and compilation of this report which is entirely based on the AS 1726-1993 – Geotechnical Investigations.

INVESTIGATIONS METHODS

Test Pits

Test pits are usually excavated with a backhoe or an excavator, allowing close examination of the in-situ soil if it is safe to enter into the pit. The depth of excavation is limited to about 3m for a backhoe and up to 6m for a large excavator. A potential disadvantage of this investigation method is the larger area of disturbance to the site. Samples can be taken from the test pits for soils testing and analyses.

Large Diameter Augers

Boreholes can be drilled using a rotating plate or short spiral auger, generally 3000mm or large in diameter commonly mounted on a standard piling rig. The cuttings are returned to the surface at intervals (generally not more than 0.5m) and are disturbed but usually unchanged in moisture content. Identification of soil strata is generally much more reliable than with continuous spiral flight augers, and is usually supplemented by occasional undisturbed tube samples.

Continuous Spiral Flight Augers

The borehole is advanced using 90-125mm diameter continuous spiral flight augers which are withdrawn at intervals to allow sampling or in-situ testing. This is a relatively economical means of drilling in clays and sands above the water table. Samples are returned to the surface, or may be mixed with soils from the sides of the hole. Information from the drilling (as a distinct from specific sampling by SPTs or undisturbed samples) is of relatively low reliability, due to the remoulding, possible mixing or softening of samples by groundwater.

Dynamic Cone Penetrometer Tests

Dynamic penetrometer tests (DCP) are carried out by driving a steel rod into the ground using a standard weight of hammer falling a specified distance. As the rod penetrates the soil the number of blows required to penetrate each successive 300mm depth are recorded. Normally there is a depth limitation of 1.2m, but this may be extended in certain conditions by the use of extension rods. A 16mm diameter rod with a 20mm diameter cone end is driven using a 9kg hammer dropping 510mm (AS 1289, Test 6.3.2). This test was developed initially for pavement subgrade investigations, and correlations of the test results with California Bearing Ratio have been published by various road authorities. Also Correlations with SPT tests can be made for Cohesion less and cohesive soils.

Standard Penetration Tests

Standard penetration tests (SPT) are used as a means of estimating the density or strength of soils and also of obtaining a relatively undisturbed sample. 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 under the impact of a 63kg hammer with a free fall of 760mm. It is normal for the tube to be driven in three successive 150mm increments equal to 450mm in total. The first 150mm increment is not considered for the so-called “N” value (standard penetration resistance), which is taken from the number of blows of the last 300mm. In dense sands, very hard clays or weak rock, the full 450mm may not be practicable and the test will be discontinued. The results are represented in the following example:

- In the case where full penetration is obtained with successive blow counts for each 150mm as follow:
 - 1st Increment (150mm) = 2 blows
 - 2nd Increment (150mm) = 8 blows
 - 3rd Increment (150mm) = 15 blows
 - Representation – 2,8,15 “N” Value = 23
- In the case where the test is discontinued before the full penetration:
 - 1st Increment (150mm) = 20 blows
 - 2nd Increment (100mm) = 40 blows – test interrupted
 - 3rd Increment (150mm) = not carried – test refusal
 - Representation – 20, 40/100 mm “N” Value = 40

The results of the SPT tests can be related empirically to the engineering properties of the soils.

Correlation between DCP vs SPT for Cohesionless Soils

DCP (Blows/300mm)	SPT Value (Blows/300mm)	RELATIVE DENSITY
0-3	0-4	Very Loose
3-9	4-10	Loose
9-24	10-30	Medium Dense
24-45	30-50	Dense
>45	>50	Very Dense

Correlation Between DCP vs SPT for Cohesive Soils

DCP (Blows/300mm)	SPT Value (Blows/300mm)	CONSISTENCY
0-3	0-2	Very Soft
3-6	2-5	Soft
6-9	5-10	Medium/Firm
9-21	10-20	Stiff
21-36	20-40	Very Stiff
>36	>40	Hard

Continuous Diamond Core Drilling

A continuous core sample can be obtained using a diamond tipped core barrel, usually with a 50mm internal diameter. Provided full core recovery is achieved (which is not always possible in weak rocks and granular soils), this technique provides a very reliable method of investigation.

Sampling

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

Disturbed samples taken during drilling provide information on colour, type, inclusions and, depending upon the degree of disturbance, some information on strength and structure.

Undisturbed samples are taken by pushing a thin-walled sample tube into the soil and withdrawing it to obtain a sample of the soil in a relatively undisturbed state. Such samples yield information on structure and strength, and are necessary for laboratory determination of shear strength and compressibility. Undisturbed sampling is generally affective only in cohesive soils.

DESCRIPTION AND CLASSIFICATIONS METHODS FOR SOILS AND ROCK

Descriptions include strength or density, colour, structure, soil or rock type and inclusions.

SOIL DESCRIPTIONS

Soil types are described according to the predominant particle size, qualified by the grading of other particles present:

Type	Particle size (mm)
Boulder	>200
Cobble	63 – 200
Gravel	0.6 – 63
Sand	0.075 – 0.6
Silt	0.002 – 0.075
Clay	<0.002

Type	Sand & Gravel Particle size
Coarse gravel	36mm – 19mm
Medium gravel	19mm – 6.7mm
Fine gravel	6.7mm – 2.36mm
Coarse sand	2.36mm – 600µm
Medium sand	600µm – 212µm
Fine sand	212µm – 75µm

The proportions of secondary constituents of soils are described as:

Coarse grained soils		Fine grained soils	
%Fines	Modifier	%Coarse	Modifier
≤ 5	Omit, or use 'trace'	≤ 15	Omit, or use 'trace'
$>5 - \leq 12$	Describe as 'with clay/silt' as applicable	$>15 - \leq 30$	Describe as 'with clay/silt' as applicable
>12	Describe as 'with silty/clayey' as applicable	>30	Describe as 'with silty/clayey' as applicable

Definitions of grading terms used are:

- Well graded – a good representation of all particle sizes.
- Poorly graded – an excess or deficiency of particular sizes within specified range.
- Uniformly graded – an excess of a particular particle size.
- Gap graded – a deficiency of a particular particle size with the range.

Cohesive Soils

Cohesive soils, such as clays, are classified on the basics of undrained shear strength. The strength may be measured by laboratory testing, or estimated by field tests or engineering examination. The strength terms are defined as follows:

Description	Abbreviation	Undrained shears strength (kPa)
Very soft	vs	≤ 12
Soft	s	$>12 - \leq 25$
Firm	f	$>25 - \leq 50$
Stiff	st	$>50 - \leq 100$
Very stiff	vst	$>100 - \leq 200$
Hard	h	>200

Cohesionless Soils

Cohesionless soils, such as clean sands, are classified on the basics of relative density, generally from the results of standard penetration tests (SPT), cone penetration tests (CPT), or dynamic penetrometers (PSP). The relative density terms are given below:

Relative density	Abbreviation	Density index %
Very loose	vl	≤ 15
Loose	l	$>15 - \leq 35$
Medium dense	md	$>35 - \leq 65$
Dense	d	$>65 - \leq 85$
Very dense	vd	>85

Soil Origin

It is often difficult to accurately determine the origin of a soil. Soils can generally be classified as:

- Residual soil – derived from in-situ weathering of the underlying rock.
- Transported soils – formed somewhere else and transported by nature to the site.
- Filling – moved by man.

Transported soils may be further subdivided into:

- Alluvium – river deposits.
- Lacustrine – lake deposits.
- Aeolian – wind deposits.
- Littoral – beach deposits.
- Estuarine – tidal river deposits.
- Talus – coarse colluvium.
- Slopwash or Colluvium – transported downslope by gravity assisted by water. Often includes angular rock fragments and boulders.

ROCK DESCRIPTIONS

Rock Strength

Rock strength is defined by the Point Load Strength (Is_{50}) and refers to the strength of the rock substance and not the strength of the overall rock mass, which may be considerably weaker due to defects. The test procedure is described by Australian Standards 1726. The terms used to describe rocks strength are as follow:

Term	Abbreviation	Point Load Index $Is_{(50)}$ MPa	Approx. Unconfined Compressive Strength MPa*
Extremely low	EL	≤ 0.03	< 0.6
Very low	VL	$> 0.03 - \leq 0.1$	$0.6 - 2$
Low	L	$> 0.1 - \leq 0.3$	$2 - 6$
Medium	M	$> 0.3 - \leq 1.0$	$6 - 20$
High	H	$> 1 - \leq 3$	$20 - 60$
Very high	VH	$> 3 - \leq 10$	$60 - 200$
Extremely high	EH	> 10	> 200

*Assumes a ratio of 20:1 for UCS to $Is_{(50)}$

Degree of Weathering

The degree of weathering of rocks is classified as follows:

Term	Abbreviation	Description
Residual	RS	Soil developed on extremely weathered rock; the mass structure and substance are no longer evident.
Extremely weathered	XW	Rock is weathered to such an extent that it has 'soil' properties, i.e. it either disintegrates or can be remoulded in water, but the texture of the original rock is still evident.
Distinctly weathered	DW	Staining and discolouration of rock substance has taken place.
Slightly weathered	SW	Rock substance is slightly discoloured but shows little or no change of strength from fresh rock.
Fresh	FR	No signs of decomposition or staining.

Degree of Fracturing

The following classification applies to the spacing of natural fractures in diamond drill cores. It includes bedding plane partings, joints and other defects, but excludes drilling breaks.

Term	Description
Fragmented	Fragments of $< 20\text{mm}$
Highly fragmented	Core lengths of $20 - 40\text{mm}$ with some fragments
Fractured	Core lengths of $40 - 200\text{mm}$ with some shorter and longer sections
Slightly Fractured	Core lengths of $200 - 400\text{mm}$ with some shorter and longer sections
Unbroken	Core lengths mostly $> 1000\text{mm}$

Rock Quality Designation

The quality of the cored rock can be measured using the Rock Quality Designation (RQD) index, defined as:

$$RQD \% = \frac{\text{cumulative length of 'sound' core sections} \geq 100\text{mm long}}{\text{total drilled length of section being assessed}}$$

Where 'sound' rock is assessed to be rock of low strength or better. The RQD applies only to natural fractures. If the core is broken by drilling or handling (i.e. drilling breaks) then the broken pieces are fitted back together and are not included in the calculation or RQD.

Rock Quality Designation

For sedimentary rocks the following terms may be used to describe the spacing of bedding partings:

Term	Separation of Stratification Planes
Thinly laminated	< 6mm
Laminated	6mm to 20mm
Very thinly bedded	20mm to 60mm
Thinly bedded	60mm to 0.2m
Medium Bedded	0.2m to 0.6m
Thickly bedded	0.6m to 2m
Very thickly bedded	> 2m

LOG SYMBOLS

Moisture Condition - Cohesive Soils:

MC > PL – Moisture content estimated to be greater than plastic limit

MC = PL - Moisture content estimated to be approximately equal to plastic limit

MC < PL - Moisture content estimated to be less than plastic limit

Moisture Condition - Cohesionless 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 less than 25 kPa

S – Soft – Unconfined compressive strength 25-50 kPa

F – Firm – Unconfined compressive strength 50-100 kPa

St – Stiff – Unconfined compressive strength 100-200 kPa

VSt – Very Stiff – Unconfined compressive strength 200-400 kPa

H – Hard - Unconfined compressive strength greater than 400 kPa

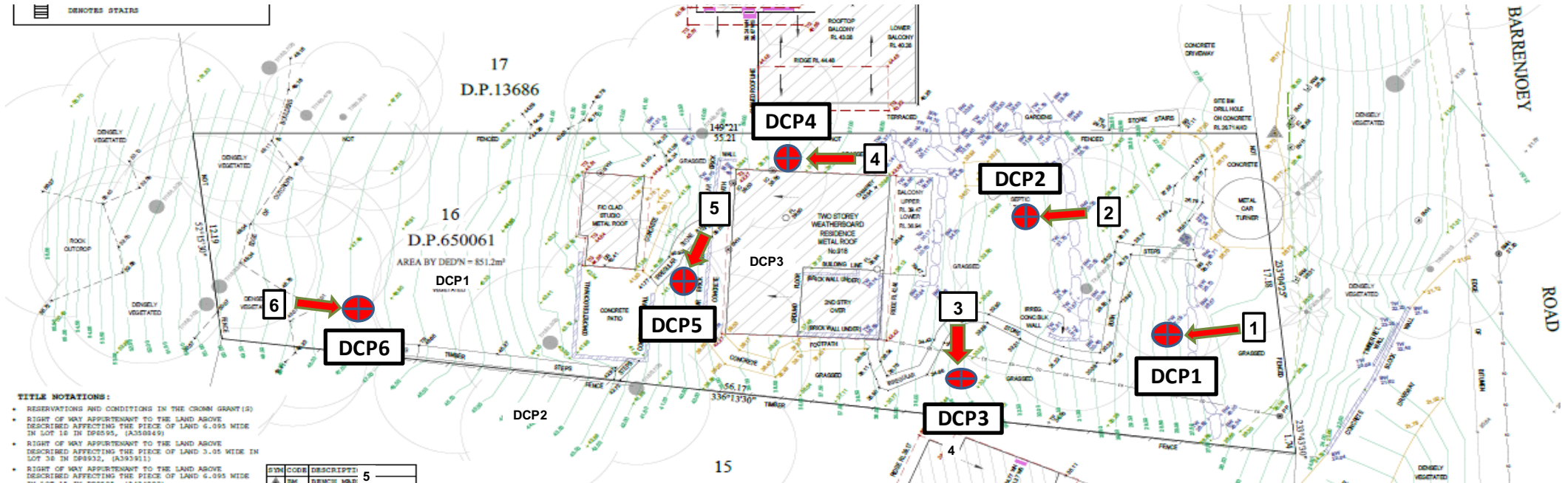
Density Index/Relative Density - Cohesionless Soils

Symbol	Density Index (ID)	Range %	SPT “N” Value Range (Blows/300mm)
VL	Very Loose	<15	0-4
L	Loose	15-35	4-10
MD	Medium Dense	35-65	10-30
D	Dense	65-85	30-50
VD	Very Dense	>85	>50

APPENDIX B

DCP TESTS & SITE PHOTOS LOCATION PLAN

918 BARRENJOY ROAD, PALM BEACHES, NSW 2108



LEGEND

-  DCP TEST LOCATION
-  PHOTO DIRECTION VIEW
- # PHOTO & DCP TEST ID

SOURCE: Survey drawing prepared by H & S LAND SURVEYORS PTY LTD, Project No; 21071, " SURVEY PLAN SHOWING DETAIL & LEVELS OVER LOT 16 IN DP 650061 NO. 918 BARRENJOY ROAD, PALM BEACHE NSW 2108, Rev A, Dated on 27-



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TITLE: DCP TESTS & SITE PHOTOS LOCATION PLAN
SITE ADDRESS: 918 BARRENJOY ROAD, PALM BEACHES, NSW 2108
CLIENT: LYNDE & DICK

Revision	Date	Date: 31-08-2021	By:	SK
		Scale: NTS	Approved:	JC
		Project No.: SRE/844/PB/21	Appendix:	B

APPENDIX C

DCP TESTS GRAPHICS



Page:	1 of 1
Date Started:	13/08/2021
Date Completed:	13/08/2021
Logged/Checked by:	SK/JC

Standards: AS 1289.6.3.2 - 1997

Figure 1 is a line graph titled "DCP blows/300mm vs Depth Plots". The vertical axis (y-axis) represents depth in meters, ranging from 0.0 at the top to 6.0 at the bottom, with major grid lines every 0.5 units. The horizontal axis (x-axis) represents DCP blows per 300mm, ranging from -5 to 65, with major grid lines every 5 units. The graph is divided into five vertical zones by red dotted lines, labeled from left to right: "VERY LOOSE" (0 to 10 blows), "LOOSE" (10 to 25 blows), "MEDIUM DENSE" (25 to 45 blows), "DENSE" (45 to 60 blows), and "VERY DENSE" (60 to 65 blows). Six data series are plotted, each representing a different DCP test location: DCP1 (orange line with triangle markers), DCP2 (light blue line with circle markers), DCP3 (purple line with diamond markers), DCP4 (brown line with square markers), DCP5 (teal line with circle markers), and DCP6 (dark purple line with diamond markers). The data points for each series are as follows:

Depth (m)	DCP1	DCP2	DCP3	DCP4	DCP5	DCP6
0.0	0	0	0	0	0	0
0.2	10	5	5	5	5	5
0.4	15	10	10	10	10	10
0.6	10	15	15	15	15	15
0.8	5	20	20	20	20	20
1.0	0	25	25	25	25	25
1.2	0	35	35	35	35	35
1.4	0	30	30	30	30	30
1.6	0	25	25	25	25	25
1.8	0	20	20	20	20	20
2.0	0	15	15	15	15	15
2.2	0	10	10	10	10	10
2.4	0	5	5	5	5	5
2.6	0	0	0	0	0	0
2.8	0	0	60	0	0	0
3.0	0	0	55	0	0	0
3.2	0	0	50	0	0	0
3.4	0	0	45	0	0	0
3.6	0	0	40	0	0	0
3.8	0	0	35	0	0	0
4.0	0	0	30	0	0	0
4.2	0	0	25	0	0	0
4.4	0	0	20	0	0	0
4.6	0	0	15	0	0	0
4.8	0	0	10	0	0	0
5.0	0	0	5	0	0	0
5.2	0	0	0	0	0	0
5.4	0	0	0	0	0	0
5.6	0	0	0	0	0	0
5.8	0	0	0	0	0	0
6.0	0	0	0	0	0	0

Figure 1 is a line graph titled "DCP blows/300mm vs Depth Plots". The y-axis represents depth in meters, ranging from 0.0 at the top to 6.0 at the bottom, with major grid lines every 0.5 units. The x-axis represents DCP blows per 300mm, ranging from -5 to 65, with major grid lines every 5 units. The graph is divided into five vertical zones by red dotted lines, labeled from left to right: "VERY LOOSE" (0 to 10 blows), "LOOSE" (10 to 25 blows), "MEDIUM DENSE" (25 to 45 blows), "DENSE" (45 to 60 blows), and "VERY DENSE" (60 to 65 blows). Six data series are plotted, each representing a different DCP test location: DCP1 (orange line with triangle markers), DCP2 (light blue line with circle markers), DCP3 (purple line with diamond markers), DCP4 (brown line with triangle markers), DCP5 (teal line with circle markers), and DCP6 (dark purple line with diamond markers). The data points for each series are as follows:

Depth (m)	DCP1	DCP2	DCP3	DCP4	DCP5	DCP6
0.0	0	0	0	0	0	0
0.2	5	3	2	1	1	1
0.4	10	5	4	3	3	3
0.6	15	8	7	6	6	6
0.8	12	10	10	10	10	10
1.0	10	12	12	12	12	12
1.2	8	15	15	15	15	15
1.4	7	18	18	18	18	18
1.6	6	20	20	20	20	20
1.8	5	22	22	22	22	22
2.0	4	25	25	25	25	25
2.2	3	28	28	28	28	28
2.4	2	30	30	30	30	30
2.6	1	32	32	32	32	32
2.8	1	33	33	33	33	33
3.0	1	34	34	34	34	34
3.2	1	35	35	35	35	35
3.4	1	35	35	35	35	35
3.6	1	35	35	35	35	35
3.8	1	35	35	35	35	35
4.0	1	35	35	35	35	35
4.2	1	35	35	35	35	35
4.4	1	35	35	35	35	35
4.6	1	35	35	35	35	35
4.8	1	35	35	35	35	35
5.0	1	35	35	35	35	35
5.2	1	35	35	35	35	35
5.4	1	35	35	35	35	35
5.6	1	35	35	35	35	35
5.8	1	35	35	35	35	35
6.0	1	35	35	35	35	35

APPENDIX D

SITE PHOTOGRAPHS



CLIENT: LYNDE & DICK
PROJECT: GEOTECHNICAL SITE INVESTIGATION REPORT FOR PROPOSED NEW DWELLING
LOCATION: 918 BARRENJOY ROAD, PALM BEACHE, NSW 2108
DATE: 31/08/2021
PROJECT NO.: SRE/844/PB/21

PAGE: 1 of 1
DATE RECORD: 13/08/2021
LOGGED BY: SK
CHECKED BY: JC

SITE PHOTOGRAPHS

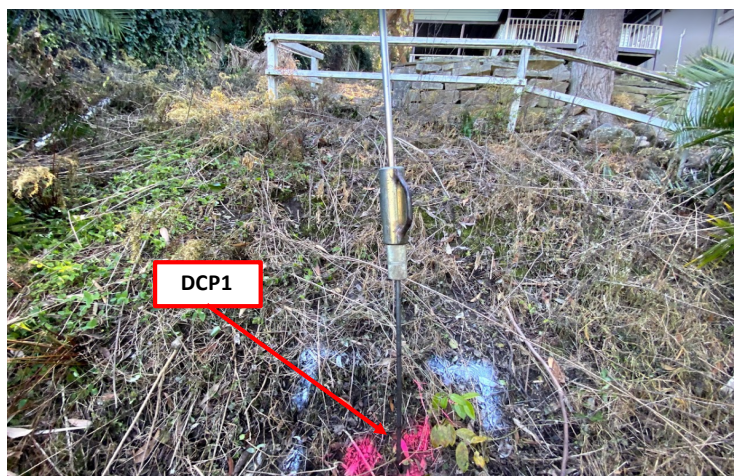


Photo 1 - Northwest view to DCP1 test location.

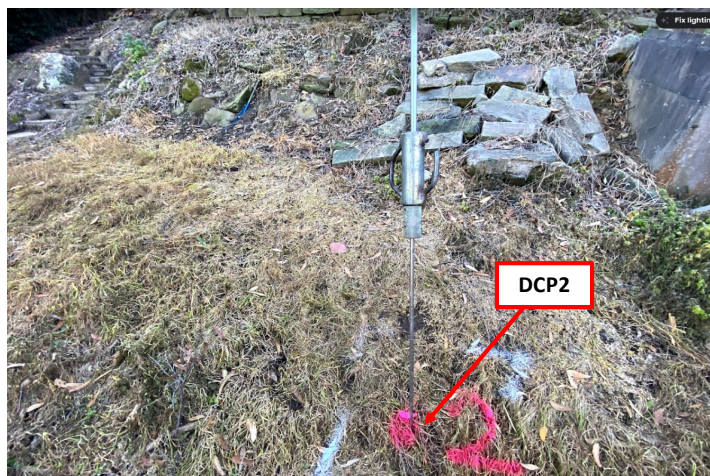


Photo 2 - Northwest view to DCP2 test location.

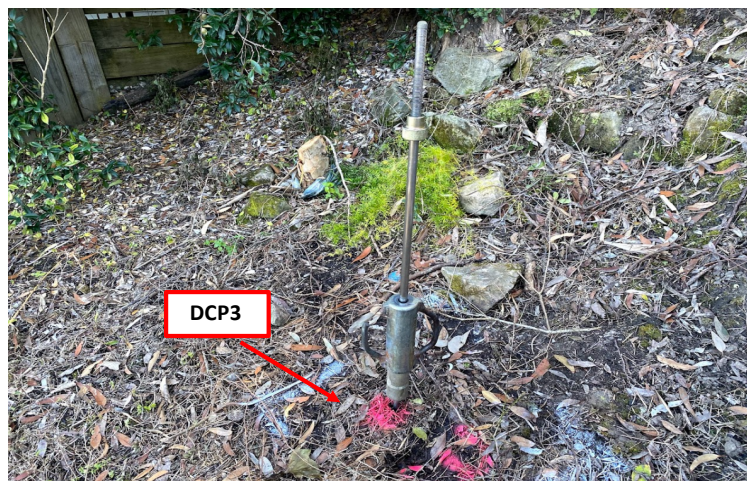


Photo 3 - West view to DCP3 test location.

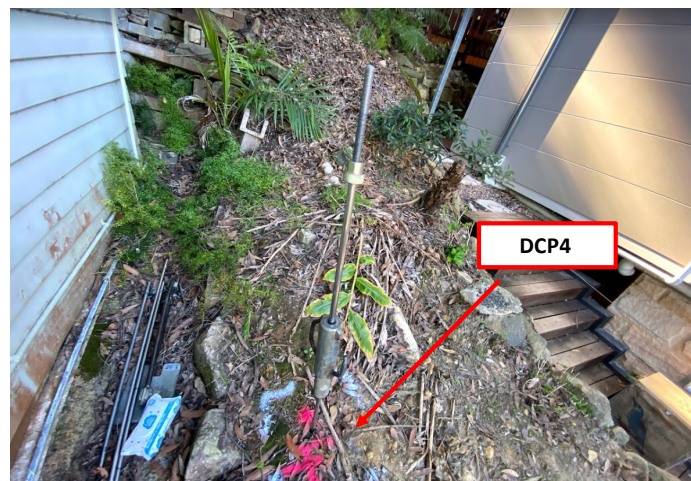


Photo 4 - North view to DCP4 test location.

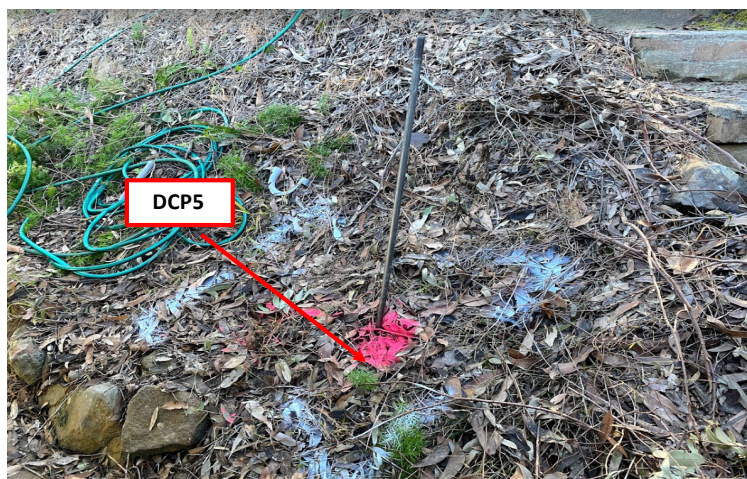


Photo 5 - Northwest view to DCP5 test location.

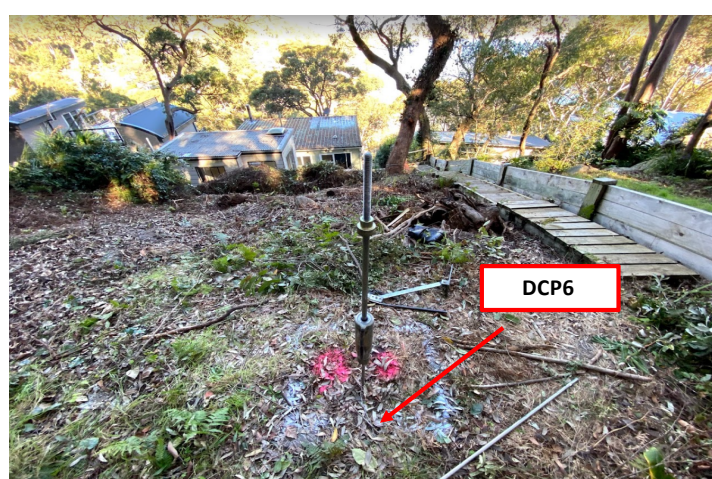


Photo 6 - South view to DCP6 test location.

APPENDIX E

LANDSLIDE RISK ASSSSMENT TERMINOLOGY FOR USE IN ASSESSING RISK TO PROPERTY (PRACTICE NOTE GUIDELINES FOR LANDSLIDE RISK MANAGEMENT 2007 AGS (AUSTRALIAN GEOMECHANICS SOCIETY)

PRACTICE NOTE GUIDELINES FOR LANDSLIDE RISK MANAGEMENT 2007
APPENDIX C: 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	5x10 ⁻²	10 years	20 years	The event is expected to occur over the design life.	ALMOST CERTAIN	A
10 ⁻²		100 years		The event will probably occur under adverse conditions over the design life.	LIKELY	B
10 ⁻³	5x10 ⁻³	1000 years	200 years	The event could occur under adverse conditions over the design life.	POSSIBLE	C
10 ⁻⁴	5x10 ⁻⁴	10,000 years	2000 years	The event might occur under very adverse circumstances over the design life.	UNLIKELY	D
10 ⁻⁵	5x10 ⁻⁵	100,000 years	20,000 years	The event is conceivable but only under exceptional circumstances over the design life.	RARE	E
10 ⁻⁶	5x10 ⁻⁶	1,000,000 years	200,000 years	The event is inconceivable or fanciful over the design life.	BARELY CREDIBLE	F

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

QUALITATIVE MEASURES OF CONSEQUENCES TO PROPERTY

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

- Notes:** (2) The Approximate Cost of Damage is expressed as a percentage of market value, being the cost of the improved value of the unaffected property which includes the land plus the unaffected structures.
- (3) The Approximate Cost is to be an estimate of the direct cost of the damage, such as the cost of reinstatement of the damaged portion of the property (land plus structures), stabilisation works required to render the site to tolerable risk level for the landslide which has occurred and professional design fees, and consequential costs such as legal fees, temporary accommodation. It does not include additional stabilisation works to address other landslides which may affect the property.
- (4) The table should be used from left to right; use Approximate Cost of Damage or Description to assign Descriptor, not *vice versa*

PRACTICE NOTE GUIDELINES FOR LANDSLIDE RISK MANAGEMENT 2007

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.

APPENDIX F

**PRACTICE NOTE GUIDELINES FOR LANDSLIDE RISK MANAGEMENT AGS 2007 AGS –
APPENDIX G – SOME GUIDELINES FOR HILLSIDE CONSTRUCTION INTRODUCTION**

PRACTICE NOTE GUIDELINES FOR LANDSLIDE RISK MANAGEMENT 2007

APPENDIX G - SOME GUIDELINES FOR HILLSIDE CONSTRUCTION

GOOD ENGINEERING PRACTICE

POOR ENGINEERING PRACTICE

ADVICE

GEOTECHNICAL ASSESSMENT	Obtain advice from a qualified, experienced geotechnical practitioner at early stage of planning and before site works.	Prepare detailed plan and start site works before geotechnical advice.
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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.
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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.	Indiscriminatory 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 property 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 rock 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 rock 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 general 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 on 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 roof runoff 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 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.

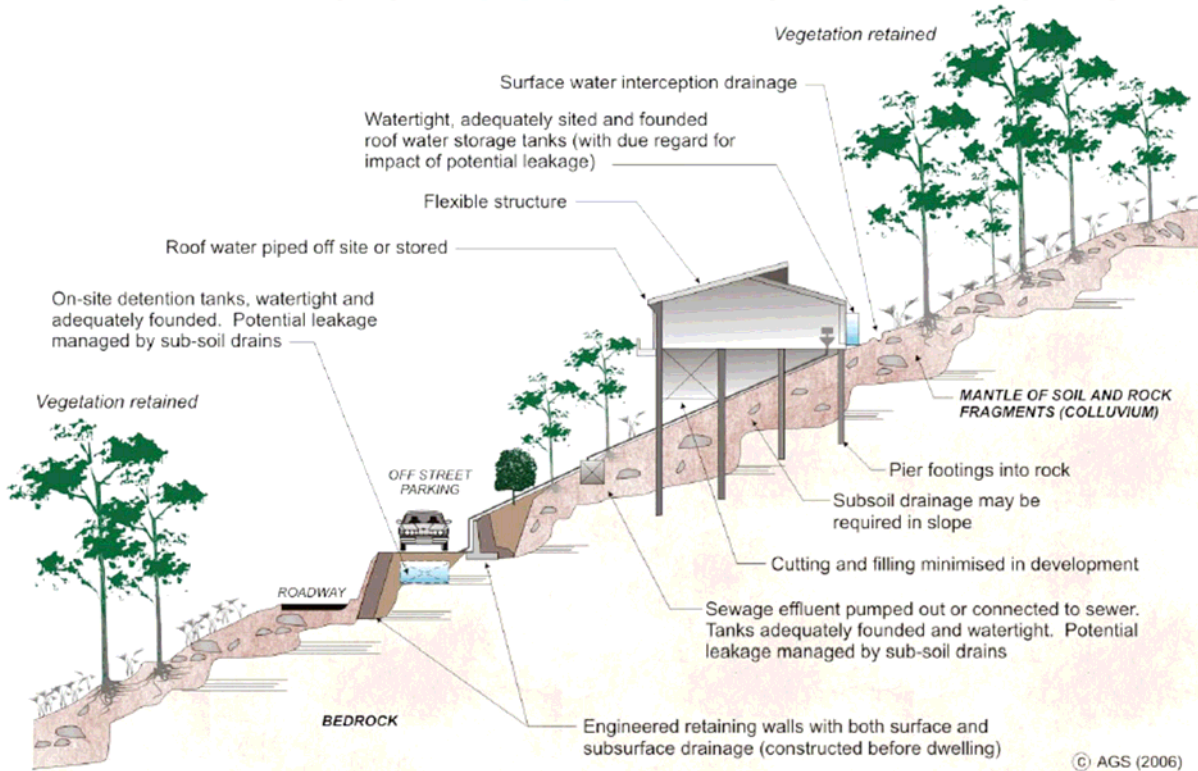
DRAWINGS AND SITE VISITS DURING CONSTRUCTION

DRAWINGS	Building Application drawings should be viewed by 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 see advice. If seepage observed, determine causes or seek advice on consequences.	
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EXAMPLES OF **GOOD** HILLSIDE PRACTICE



EXAMPLES OF **POOR** HILLSIDE PRACTICE

