Preliminary Geotechnical Assessment

201 Headland Road, Curl Curl, NSW

Final Report

P2511036JR01V02

October 2025

Prepared for Ryan and Natalia Dinsdale

environmental science & engineering

Project Details

Report Title	Preliminary Geotechnical Assessment: 201 Headland Road, Curl Curl, NSW
Client	Ryan and Natalia Dinsdale
Document	P2511036JR01V02
Director	Dr Daniel Martens
Manager	Ken Burgess
Principal Author	Ali Kandil

Document History

Issue	Issue Date	Status	Description / Comment	Author	Reviewer	Approved
1.	30/09/2025	Draft	Draft for client review	AK	KB	KB
2.	08/10/2025	Final	Updated geological cross section	AK	KB	KB

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1 Introduction & Background

1.1 Overview

Martens and Associates (**MA**) was engaged by Modify Homes Pty Ltd on behalf of Ryan and Natalia Dinsdale to carry out a preliminary geotechnical assessment in relation to the proposed development at 201 Headland Road, Curl Curl, NSW (‘**the Site**’).

This preliminary geotechnical assessment was undertaken in accordance with the scope of work outlined in MA proposal P2511036BC01V01 dated 19 August 2025.

1.2 Assessment Purpose

The purpose of this preliminary geotechnical assessment is to support a Development Application (DA) to Northern Beaches Council (**NBC**) and preliminary structural design.

As the site is mapped on the Council landslip risk map in Warringah ‘Area B’, a preliminary geotechnical assessment is required according to Warringah Development Control Plan (2011) Part *E10: Landslip Risk.*

1.3 Proposed Alterations and Additions

We understand from the architectural plans (Modify, 2025) that the proposed alterations and additions to the existing dwelling will comprise:

- Partial demolition of existing internal and external walls.
- Demolition of gazebo structure at the front of the site.
- Construction of new internal and external alterations and additions.
- Construction of proposed rumpus room (RL 53.250 mAHD).
- Extensions of the lower ground floor to accommodate the proposed bedroom 4 and new stairway access to the garage (RL 53.250 mAHD).

The maximum excavation depth for the proposed lower ground floor extension at the front of the existing building, is unknown but is estimated to extend up to appropriately 1.0 - 1.5 metres below ground level (mbgl).

2 Investigation Scope

2.1 Investigation Scope of Work

Field investigations conducted on 27 March 2025 included:

- Review of BYDA survey plans and locating of underground services.
- A Site walkover by a geotechnical engineer to assess slope risk and to review local geology, soil exposures, surface hydrology, topography, drainage and relevant site features.
- Drilling of four boreholes (BH101 to BH104) using hand methods up to a maximum depth of 1.1 mbgl (BH101).
- Three Dynamic Cone Penetrometer (DCP) tests (DCP101 to DCP103) at respective borehole locations up to 1.15 mbgl (DCP101).
- Collection of soil samples from boreholes for future reference.

Investigation locations are shown on Figure 1, Appendix A.

3 Site Details and Subsurface Conditions

3.1 General Site Details

General Site details are summarised in Table 1.

Table 1: Summary of general Site details (based on desktop reviews, site walkover and field investigations).

Item	Description / Comment
Property address	201 Headland Road, Curl Curl, NSW (‘**the Site**’).
Lot / DP	Lot 6 in DP 12515 (CMS, 2025).
LGA	Northern Beaches Council (**Council**).
Site area	838.7 m^2 (CMS, 2025).
Topography	Within highly undulating terrain, the site is located within the flank of a roughly north south aligned spur of a local radial peak to the northeast.
Typical slopes, aspect, elevation	The Site generally has a southerly aspect with near level terraces. The northern portion of the Site appears to be founding on a sandstone shelf with grades < 5 % and separated from the southern portion of the site by a near vertical sandstone rock face. The southern portion of the site grades range between 5% and 10% with steeper areas of sandstone outcrop. Ground levels across the Site range between approximately 57 mAHD in the northern portion and 47 mAHD in the southern portion of the Site (CMS, 2025).
Expected geology	Hawkesbury Sandstone, comprising medium to coarse grained quartz sandstone with very minor shale and laminite lenses (Herbert, 1983).
Expected Soil Landscape	The NSW Office of Environment and Heritage’s (**OEH**) information system (eSPADE) indicates the Site is underlain by the Gymea (gy) soil landscape, typically characterised by shallow to moderately deep clay / silty clay, with total soil depth up to approximately 1.0 m. Soil limitations include localised steep slopes, high soil erosion hazard, rock outcropping shallow, highly permeable soil, and; very low soil fertility.
Existing Development	A three storey clad house is situated in the northern portion of the site with an inground swimming pool and shed in the central portion of the site. The southern portion of the site comprises a lawn bisected by a sewer main.
Vegetation	The northern portion of the site is covered in hardstand. The southern portion of the site comprises a grassed lawn with young trees and bushes along the eastern and western boundaries.
Drainage	Via overland flow towards the south into stormwater drains.
Neighbouring environment	The Site is bordered by: • Headland Road to the north. • A three storey residential dwelling to the east. • A two storey residential dwelling to the west. • A residential dwelling to the south.

3.2 Subsurface Conditions

Field investigations revealed the following generalised subsurface units underlie the Site:

Unit A: Fill comprising poorly compacted sandy clayey silt / sand encountered up to approximately 0.94 mbgl.

Unit B: Residual deposits comprising loose to medium dense silty sand encountered up to approximately 1.1 mbgl.

Unit C: Highly weathered, inferred low strength sandstone bedrock encountered at investigation termination depths of up to 1.1mbgl.

Encountered conditions are described in more detail in the borehole logs in Appendix B. Associated explanatory notes are provided in Appendix H. For DCP test result, refer to Appendix C.

3.3 Groundwater Observations

Groundwater inflow was not encountered during drilling of boreholes up to 1.1 mbgl. Given the Site elevation, topography and encountered subsurface profile, the permanent groundwater table is unlikely to be encountered during bulk excavation. However, ephemeral perched groundwater may be encountered within the soils or at the soil / rock interface, originating from infiltration of surface water during prolonged or intense rainfall events.

Should further information on permanent site groundwater levels be required, additional investigation would need to be carried out (i.e. installation of groundwater monitoring wells and appropriate monitoring).

4 Geotechnical Assessment

4.1 Preliminary Material Properties

Material properties inferred from observations during borehole drilling, such as auger penetration resistance and DCP test results as well as engineering judgement, are summarised in Table 2.

Table 2: Soil and rock strength properties.

Layer [1]	$\gamma_{in\text{-}situ}$ [2] (kN/m^3)	C′ [3] (kPa)	Ø′ [4] (deg)	E′ [5] (MPa)	K_0 [6]	Ka [6]	K_p [6]
FILL: Sandy clayey SILT / SAND (poorly compacted)	16	0	27	4	0.55	0.38	2.64
RESIDUAL: Silty SAND (loose to medium dense)	17	0	30	10	0.50	0.33	3.0
WEATHERED ROCK: Sandstone (highly weathered, inferred low strength)	22	25	30	75	0.50	0.33	3.0

Notes:

[1] Refer to borehole logs in Appendix B for material description details.

[2] Material in-situ unit weight estimate.

[3] Average drained cohesion estimate.

[4] Average effective internal friction angle estimate assuming drained conditions; may be dependent on rock defect conditions.

[5] Effective elastic modulus estimate.

[6] K_0 = Earth pressure coefficient at rest; K_a = Active earth pressure coefficient; K_p = Passive earth pressure coefficient.

4.2 Geotechnical Landslip Risk Assessment

4.2.1 Overview

In accordance with Warringah Development Control Plan (**DCP**) Part E10: Landslip Risk, the site would be classified as Warringah "Area B". The geotechnical report required for submission with the Development Application (DA), is required to include a risk assessment in relation to both property and life.

4.2.1 Site Observations

The site walkover revealed the following:

- No evidence of recent or former large – scale gross slope instability (i.e. landslip) was observed within the site and surrounding land.
- Slopes in the northern portion of the site were < 5% and are assumed to coincide with the near level profile of sandstone terraces.
- A near vertical sandstone shelf was observed beneath the staircase along the eastern site boundary.
- Slopes in the central and southern portion of the site range between 5% and 10%

- Numerous areas of rock outcrop were observed in the central and southern portion of the site.
- The existing 3 storey dwelling was supported by piers inferred to extend into underlying bedrock.
- A detached / semi-buried sandstone boulder was exposed in the central portion of the site along the western boundary (refer to Appendix D for photos).
- No evidence of leaning trees was observed within or surrounding the site
- No evidence of water-logged soil was observed across the site.

4.2.2 Hazard Assessment

A geotechnical hazard risk assessment for the proposed works has been completed in accordance with the qualitative risk matrices provided in Section 7 of the Australian Geomechanics Society's *Landslide Risk Management Guidelines* (2007). Two slope movement mechanisms (i.e. soil creep and boulder roll rock fall) are considered most likely to impact the proposed development and existing structures on and in the immediate vicinity of the site (see Appendix E – Geotechnical Risk Calculation Sheet).

An idealised geotechnical section with annotated slope failure mechanisms is provided in Figure 2, Appendix A.

4.2.3 Conclusion

The proposed development is considered to constitute an acceptable risk to life and a low risk to property, resulting from assessed geotechnical hazard, provided that the slope treatment measures presented in Appendix E and recommendations presented in this report are adhered to. "Some Guidelines For Hillside Construction" are provided in Appendix F.

5 Geotechnical Recommendations

5.1 Overview

Geotechnical recommendations for site development are provided below. Further general geotechnical recommendations are provided in Appendix G.

5.2 Excavation and Vibrations

Proposed excavations will likely be through fill and residual soils followed by inferred very low and low to medium strength bedrock. Considering ground conditions, the following excavation plant may be required:

- Soils and very low strength bedrock should be readily excavated using conventional earthmoving equipment. Higher strength bands may require ripping tyne (or similar) to penetrate.
- Low (or higher) strength rock (if encountered): Hydraulic earthmoving equipment with rock hammer attachment.

We recommend using rock sawing techniques prior to the use of hydraulic hammer equipment (if needed) to reduce noise and ground vibrations. However, the use of vibration inducing equipment (e.g. concrete breaker, rock hammer etc.) will require vibration management in accordance with AS2187.2, Appendix J to minimise adverse impacts on the adjacent properties.

All excavation work should be completed with reference to the most recent version of Code of Practice 'Excavation Work', by Safe Work Australia.

5.3 Excavation Support

All excavations must be temporarily and permanently battered back / supported / retained to maintain excavation stability and limit potential adverse impacts on surrounding structures / neighbouring properties. Appropriate support methodologies should be adopted by the excavation contractor and design engineer and approved by a geotechnical engineer.

5.3.1 Temporary Batter Slopes

Where there is sufficient setback between bulk excavation and adjacent structures or site boundaries, excavations into soil and bedrock may be temporarily battered back at 1V:2H and 1V:1H respectively, subject to inspection and approval by an experienced geotechnical engineer.

Temporary batters should be protected from erosion and have adequate drainage to divert surface water away from the slope and prevent accumulation at the toe and crest. It is assumed that the temporary excavation batters will remain unsupported for no more than two months. Surcharge loads (e.g., buildings, roads, plant, etc.) should not be present within 1.5 m of the batter slope crest.

5.3.2 Temporary Shoring

Where there is insufficient setback between bulk excavation and site boundaries or where adjacent structures are within the zone of influence of the excavation, we recommend temporary shoring be adopted to provide support during excavation. This may include closely spaced soldier pile wall or contiguous pile wall providing that the wall is sufficiently robust to support any adjacent loads. Temporary support systems may also be designed to be incorporated into the permanent retention structure.

Design of all retaining structures should consider additional surcharge loading from live loads, new and existing structures, construction equipment, backfill compaction, sloping ground and hydrostatic pressures (if applicable) behind retaining walls unless subsurface drainage behind retaining walls are provided.

5.4 Foundations

Structural loads should be supported by foundation in general accordance with good engineering practice for hillside construction as provided in Appendix G of AGS guidelines (see Appendix F) and comprise shallow foundations embedded at least 0.5 m into bedrock or shallow piers socketed at least 1.0 m into at least very low strength sandstone. An allowable bearing capacity of 600 kPa and 700 kPa may be adopted for shallow and pier foundations respectively.

Bulk excavation of the extension at the front of the building, will likely expose variable strength sandstone bedrock following excavation. Suitable foundations may therefore comprise a concrete slab with slab thickening for strip or pad footings. Where foundations are close to sloping ground, consideration should be given to extending dowels bars into bedrock and into shallow footings / piers, to provide additional lateral resistance as needed.

All foundations must be founded within consistent materials / conditions to limit differential movements and should be designed by a suitably qualified and experienced structural engineer. Inspections should be undertaken during construction by a geotechnical engineer to confirm suitable socket depth has been achieved and design assumptions are satisfied.

5.5 Rock boulder / block stabilisation

Boulders (where present) were generally obscured by vegetation. It is therefore recommended that following site clearance, the stability of any loose boulders and detached rock blocks, should be assessed by a geotechnical engineer to determine the requirements for stabilisation or removal.

5.6 Drainage Requirements

All site discharges should be passed through a filter material prior to release. Diverted flows should be directed (where possible) to a suitable stormwater system to prevent water accumulating in areas surrounding retaining structures and footings.

5.7 Soil Erosion Control

Removal of soil overburden should be performed in a manner that reduces the risk of sedimentation occurring in the Council stormwater system and on neighbouring lands. All spoil on site should be properly controlled by erosion control measures to prevent transportation of sediments off-site. Appropriate soil erosion control methods in accordance with Landcom (2004) shall be required.

5.8 Site Classification

The site is classified as a Class ‘P’ site in accordance with AS 2870 (2011), due to sloping ground and the presence of uncontrolled fill across the site.

6 Proposed Additional Works

6.1 Further Works

We recommend the following additional geotechnical investigation and assessment works are carried out at the construction certificate (CC) stage of work to develop the final design prior to construction:

- Additional geotechnical assessment should be carried out to identify any loose boulders that may require stabilisation and / or further geotechnical advice
- Development and implementation of a geotechnical monitoring plan to provide suitable monitoring during construction including location of instruments and trigger levels, should excavation into medium or higher strength bedrock be required.
- Review of the final design and construction staging plans by a senior geotechnical engineer to confirm adequate consideration of the geotechnical risks and adoption of the recommendations provided in this report.

7 References

Australian Geomechanics Society (2007) *Landslide Risk Management*, Australian Geomechanics 42 (1).

CMS Surveyors (2025), *BOUNDARY IDENTIFICATION AND DETAIL & LEVEL SURVEY OVER LOT 6 IN DP12515 No.201 HEADLAND ROAD NORTH CURL CURL, NSW, 2099.* Drawing Name: 22546Adetail, Revision: 1, Dated 04.06.2025 (CMS, 2025)

Herbert C. (1983) Sydney 1:100 000 Geological Sheet 9130, 1st edition, Geological Survey of New South Wales, Sydney.

Landcom (2004) *Managing Urban Stormwater: Soils and Construction.*

Modify (2025), *201 Headland Road, North Curl Curl, NSW 2099,* Drawing No(s). DA00, DA10-13, DA20-23 and DA30, Revision A, Dated 27.08.2025. (Modify, 2025)

NSW Department of Environment & Heritage (eSPADE, NSW soil and land information), www.environment.nsw.gov.au.

Standards Australia Limited (1997) AS 1289.6.3.2:1997, *Determination of the penetration resistance of a soil – 9kg dynamic cone penetrometer test*, SAI Global Limited.

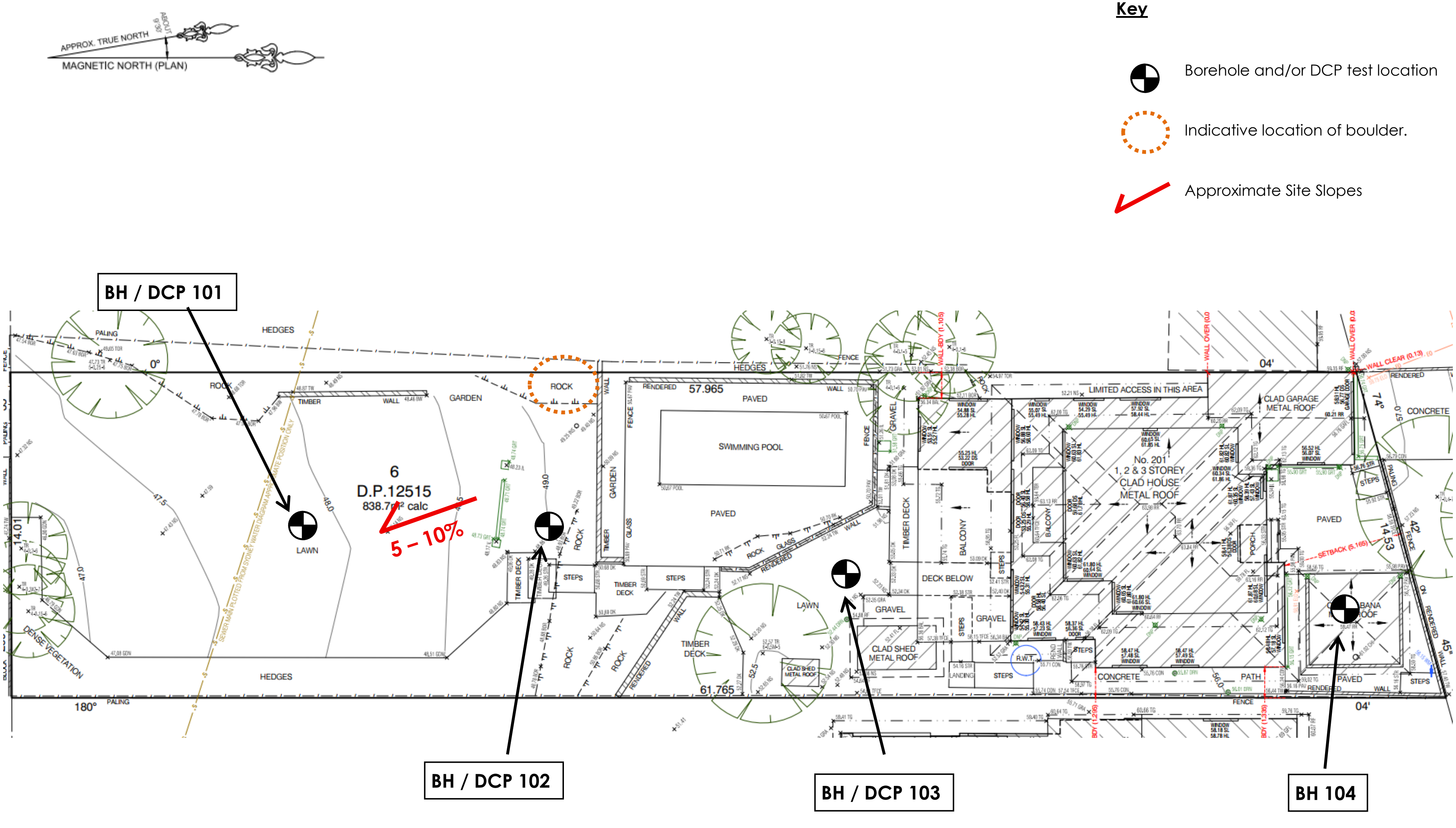
Standards Australia Limited (2006) AS 2187.2:2006, Explosives—Storage and use, SAI Global Limited.

Standards Australia Limited (2011) AS 2870:2011, *Residential slabs and footings*, SAI Global Limited.

Standards Australia Limited (2017) AS 1726:2017, *Geotechnical site investigations*, SAI Global Limited.

Warringah Council (2021) *Development Control Plan, Part E: the Natural Environment*

8 Appendix A – Figures



Key

Borehole and/or DCP test location

Indicative location of boulder.

Approximate Site Slopes

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Martens & Associates Pty Ltd ABN 85 070 240 890		Environment | Water | Wastewater | Geotechnical | Civil | Management	
Drawn:	AK	**GEOTECHNICAL TESTING PLAN** **Preliminary Geotechnical Assessment** **201 Headland Road, Curl Curl, NSW** (Source: CMS, 2025)	Drawing: **FIGURE 1**
Approved:	KB		
Date:	30.09.2025		
Scale:	NA		File No: P2511036JR01

Key:

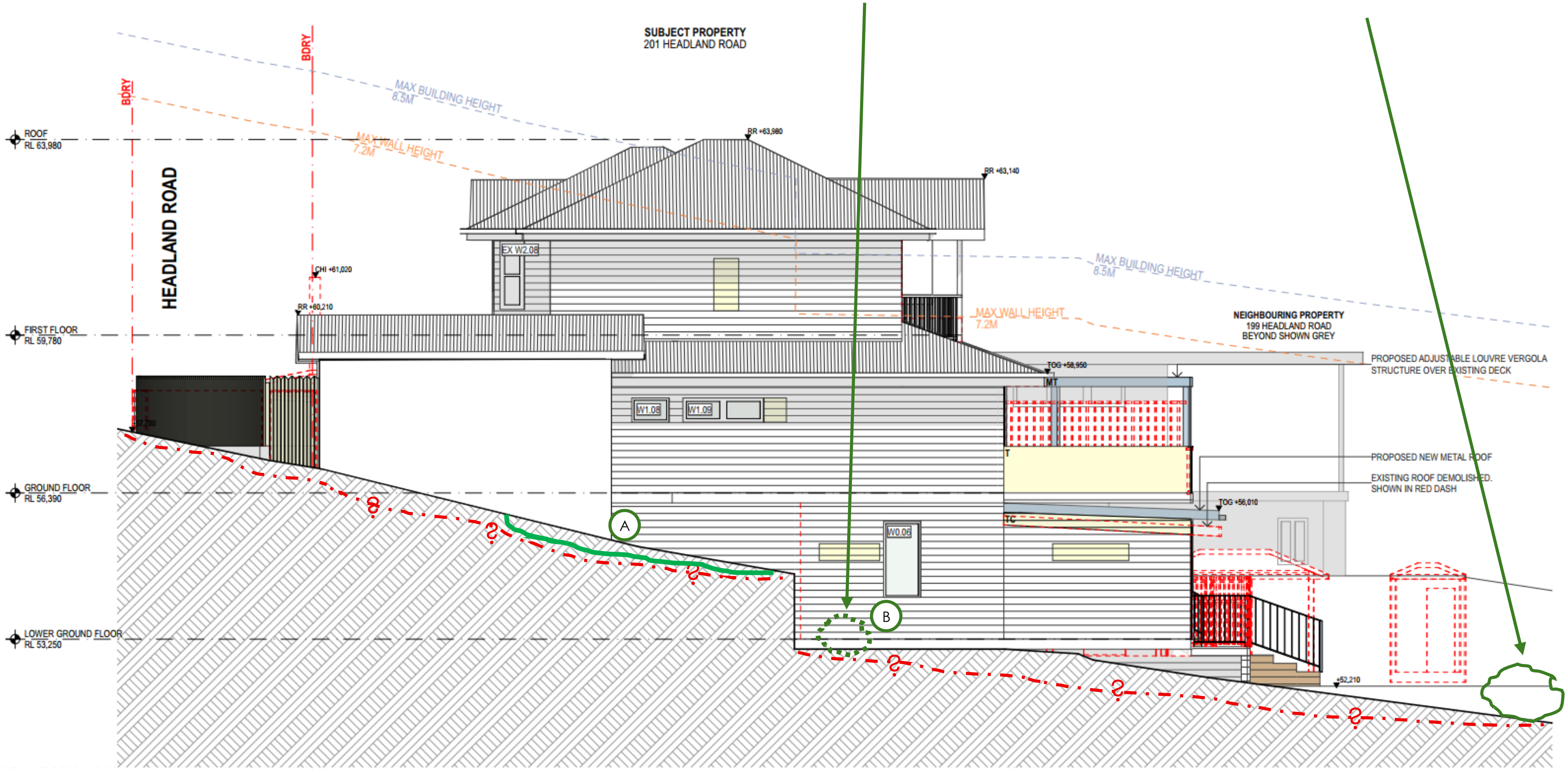
Translational Earth Creep: A

Boulder Roll: B

Indicative Rock Profile

Possible loose boulders, indicatively marked due to limited access during inspection.

Existing boulder (inferred) observed along the western boundary, approximately 16 m from the edge of the existing deck.



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Environment | Water | Wastewater | Geotechnical | Civil | Management

Drawn:	AK
Approved:	KB
Date:	08.10.2025
Scale:	Not to Scale

TYPICAL GEOLOGICAL CROSS SECTION
Preliminary Geotechnical Assessment
201 Headland Road, Curl Curl, NSW
(Source: Modify, 2025)

Drawing: **FIGURE 2**

File No: P2511036JR01

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9 Appendix B – Borehole Logs

CLIENT	Ryan and Natalia Dinsdale	COMMENCED	10/09/2025	COMPLETED	10/09/2025	**REF BH101**	
PROJECT	Preliminary Geotechnical Assessment	LOGGED	Ali Kandil	CHECKED	KB		
SITE	201 Headland Rd, North Curl Curl, NSW	GEOLOGY	Hawkesbury	VEGETATION	Grass	PROJECT NO. P2511036	
EQUIPMENT	Push Tube	LONGITUDE	151.28318	RL SURFACE	47.8	DATUM	56H
OPERATOR	JRW	LATITUDE	-33.76052	ASPECT	South	SLOPE	<10%

Drilling Method	Penetration Resistance	Water	Depth (m)	Elevation / Depth (m)	Sampling	Testing	Recovery	Graphic Log	USCS Classification	Soil / Rock Material Description	Moisture	Consistency	Soil Origin
PT		NO	0.5		D / 0.1-0.2 D / 0.4-0.5				ML	FILL: Clayey Sandy SILT; low plasticity; dark grey; trace fine to medium sized sandstone gravels; inferred poorly compacted.	w < PL	Pc	Fill
			1.0	47.2 0.6	D / 0.6-0.8 D / 0.9-1.0				SM	Silty SAND; medium to coarse grained; grey & dark grey; trace fine sized sandstone gravels.	W-M	L-Md	Residual
										BH101 Refusal at 1.1 m (on inferred sandstone bedrock.)			

EXCAVATION LOG TO BE READ IN CONJUNCTION WITH ACCOMPANYING REPORT NOTES AND ABBREVIATIONS

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Engineering Log - BOREHOLE

CLIENT	Ryan and Natalia Dinsdale	COMMENCED	10/09/2025	COMPLETED	10/09/2025	**REF BH102**	
PROJECT	Preliminary Geotechnical Assessment	LOGGED	Ali Kandil	CHECKED	KB		
SITE	201 Headland Rd, North Curl Curl, NSW	GEOLOGY	Hawkesbury	VEGETATION	Grass	PROJECT NO. P2511036	
EQUIPMENT	Push Tube	LONGITUDE	151.28322	RL SURFACE	49	DATUM	56H
OPERATOR	JRW	LATITUDE	-33.76042	ASPECT	South	SLOPE	<10%

Drilling Method	Penetration Resistance	Water	Depth (m)	Elevation / Depth (m)	Sampling	Testing	Recovery	Graphic Log	USCS Classification	Soil / Rock Material Description	Moisture	Consistency	Soil Origin
PT		NO			D / 0.1-0.2 D / 0.2-0.5				ML	FILL: Clayey Sandy SILT; low plasticity; dark grey; with fine to coarse sized sandstone gravels; inferred poorly compacted.	w < PL	Pc	Fill
			0.5	48.5 0.5	D / 0.5-0.6				SM	Silty SAND; medium to coarse grained; grey; trace fine sized sandstone gravels.	W	L-Md	Residual
										BH102 Refusal at 0.65 m (on inferred sandstone bedrock.)			

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Engineering Log - BOREHOLE

CLIENT	Ryan and Natalia Dinsdale	COMMENCED	10/09/2025	COMPLETED	10/09/2025	**REF BH103**	
PROJECT	Preliminary Geotechnical Assessment	LOGGED	Ali Kandil	CHECKED	KB		
SITE	201 Headland Rd, North Curl Curl, NSW	GEOLOGY	Hawkesbury	VEGETATION	Grass	PROJECT NO. P2511036	
EQUIPMENT	Push Tube	LONGITUDE	151.28327	RL SURFACE	52.2	DATUM	56H
OPERATOR	JRW	LATITUDE	-33.76031	ASPECT	South	SLOPE	<10%

Drilling Method	Penetration Resistance	Water	Depth (m)	Elevation / Depth (m)	Sample and/or Field Test: Sampling	Sample and/or Field Test: Testing	Recovery	Graphic Log	USCS Classification	Soil / Rock Material Description	Moisture	Consistency	Soil Origin
PT		NO			D / 0.1-0.3				SC-SM	FILL: Clayey Silty SAND; fine to medium grained; dark grey & brown; trace fine sized sandstone gravels; inferred poorly compacted.	M	Pc	Fill
			0.5										
				51.5 / 0.7	D / 0.7-0.8					Gravels becoming with; clay content decreasing.			
										BH103 Refusal at 0.94 m (on inferred sandstone bedrock.)			

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Engineering Log - BOREHOLE

CLIENT	Ryan and Natalia Dinsdale	COMMENCED	10/09/2025	COMPLETED	10/09/2025	REF BH104
PROJECT	Preliminary Geotechnical Assessment	LOGGED	Ali Kandil	CHECKED	KB	
SITE	201 Headland Rd, North Curl Curl, NSW	GEOLOGY	Hawkesbury	VEGETATION	nil	PROJECT NO. P2511036
EQUIPMENT	Push Tube	LONGITUDE	151.28333	RL SURFACE	56	DATUM 56H
OPERATOR	JRW	LATITUDE	-33.76012	ASPECT	South	SLOPE <10%

Drilling Method	Penetration Resistance	Water	Depth (m)	Elevation / Depth (m)	Sample and/or Field Test: Sampling	Sample and/or Field Test: Testing	Recovery	Graphic Log	USCS Classification	Soil / Rock Material Description	Moisture	Consistency	Soil Origin
Diamond Core		NO							CCT	Pavement Concrete ; tiles above.			Non-Soil
										BH104 Refusal at 0.26 m (on inferred sandstone bedrock.)			

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Engineering Log - BOREHOLE

10 Appendix C – DCP Test Results

Dynamic Cone Penetrometer Test Log Summary

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Site	201 Headland Road, Curl Curl, NSW	**DCP Group Reference**	P2511036JS01V01
Client	Ryan and Natalia Dinsdale	**Log Date**	10/09/2025
Logged by	AK		
Checked by	KB		
Comments	Testing commenced 50mm below ground level.		

TEST DATA

Depth Interval (m)	DCP 101	DCP 102	DCP 103					
0.15	1	HW	HW					
0.30	1	1	1					
0.45	3	24	1					
0.60	2	3	1					
0.75	4 / 100 mm	5	1					
0.90	Termiated at 0.75 mbgl due to hammer bounce.	4	3 / 140 mm					
1.05		5	Termiated at 0.94 mbgl due to hammer bounce.					
		7 / 50 mm						
		Termiated at 1.15 mbgl due to hammer bounce.						

HW = Hammer weight

11 Appendix D – Site Photographs

Photo 1: Detached / semi-buried sandstone boulder, exposed in the central portion of the site along the western boundary.

12 Appendix E – Geotechnical Risk Calculation Sheet

Slope Instability Risk - Summary Assessment

Method based on Walker et al. in AGS Vol 42 No. 1 March 2007
Method ST-38 V02 Revised 27.05.2020

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PROJECT DETAILS

Client:	Ryan and Natalia Dinsdale			Ref. No.	P2511036
Project:	Preliminary Geotechnical Assessment	Author:	AK		24.09.2025
e Address:	201 Headland Road, Curl Curl, NSW	Reviewer:	KB	Date Reviewed	24.09.2025

RISK ASSESSMENT

Risk	Hazard Type	Likelihood [1]	Consequence [1]	Risk to Life [1] Probability	Risk to Life [1] Assessment	Risk to Property [1] Likelihood	Risk to Property [1] Consequence	Risk to Property [1] Assessment
A	Translational Earth Creep	Possible	Insignificant	3.11E-08	Lr-A	Possible	Insignificant	VL
B	Boulder Roll	Unlikely	Minor	1.80E-08	Lr-A	Unlikely	Minor	L

Notes

1. Assumes treatment measures are adopted.

Definitions

1. Risk to Life Assessment - Lr-A: Acceptable risk for loss of life for the person(s). Risk level suitable for new developments.
2. Risk to Life Assessment - Lr-T: Tolerable risk for loss of life for the person(s). Risk level suitable for existing structures > 10 years old. Risk level unsuitable for new developments.
3. Risk to Life Assessment - Lr-U: Unacceptable risk for loss of life for the person(s). Risk level unsuitable for new or existing (>10 years old) developments.

Risk Level Implications

1. VH - Very High Risk - Unacceptable without treatment. Extensive detailed investigation and research, planning and implementation of treatment options essential to reduce to Low. Cost could be prohibitiv
2. H - High Risk - Unacceptable without treatment. Detailed investigation, planning and implementation of treatment options required to reduce risk to Low. Treatment will be costly.
3. M - Moderate Risk - May be tolerated in certain circumstances but requires investigation, planning and implementation to reduce risk to Low. Treatment options are practical.
4. L - Low Risk - Usually acceptable to regulators. Where treatment has been requir3ed to reduce the risk to this level, ongoing maintenance is required.
5. VL - Very Low Risk - Acceptable. Manage by normal slope maintenance procedures.

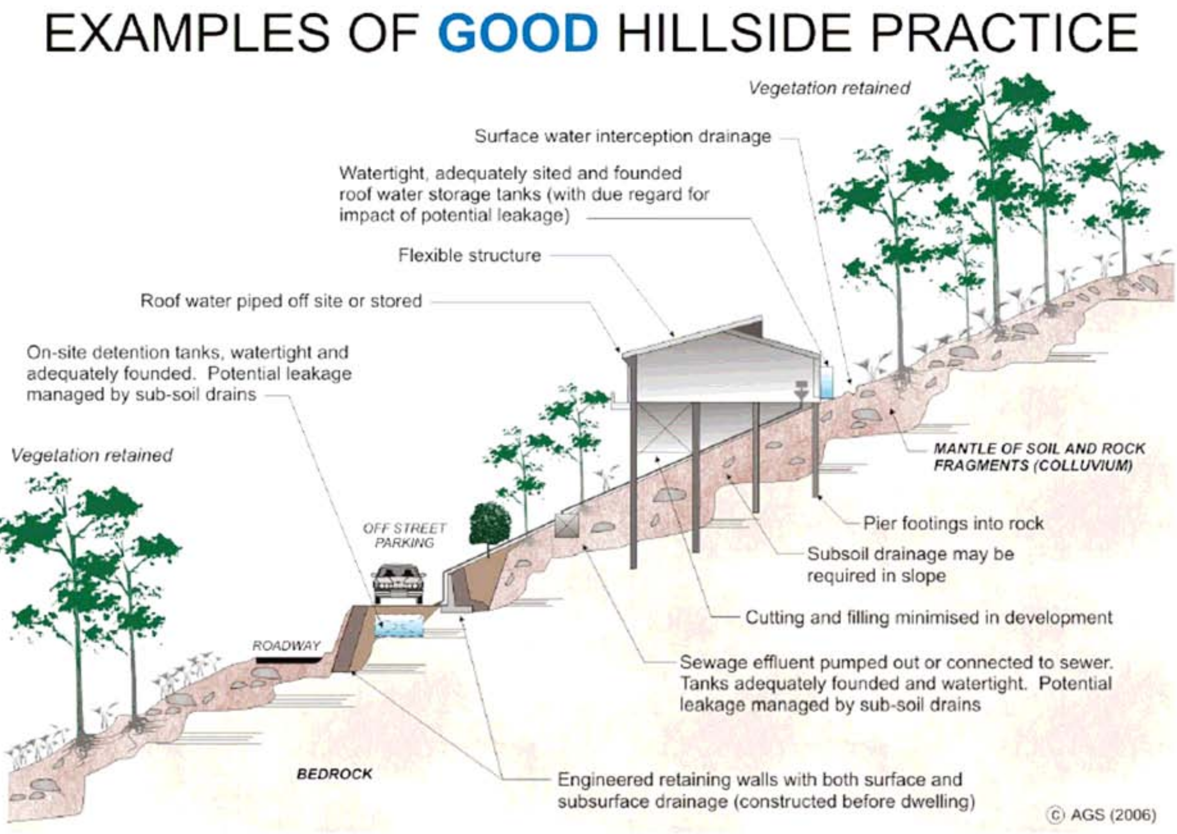
Treatment Measures

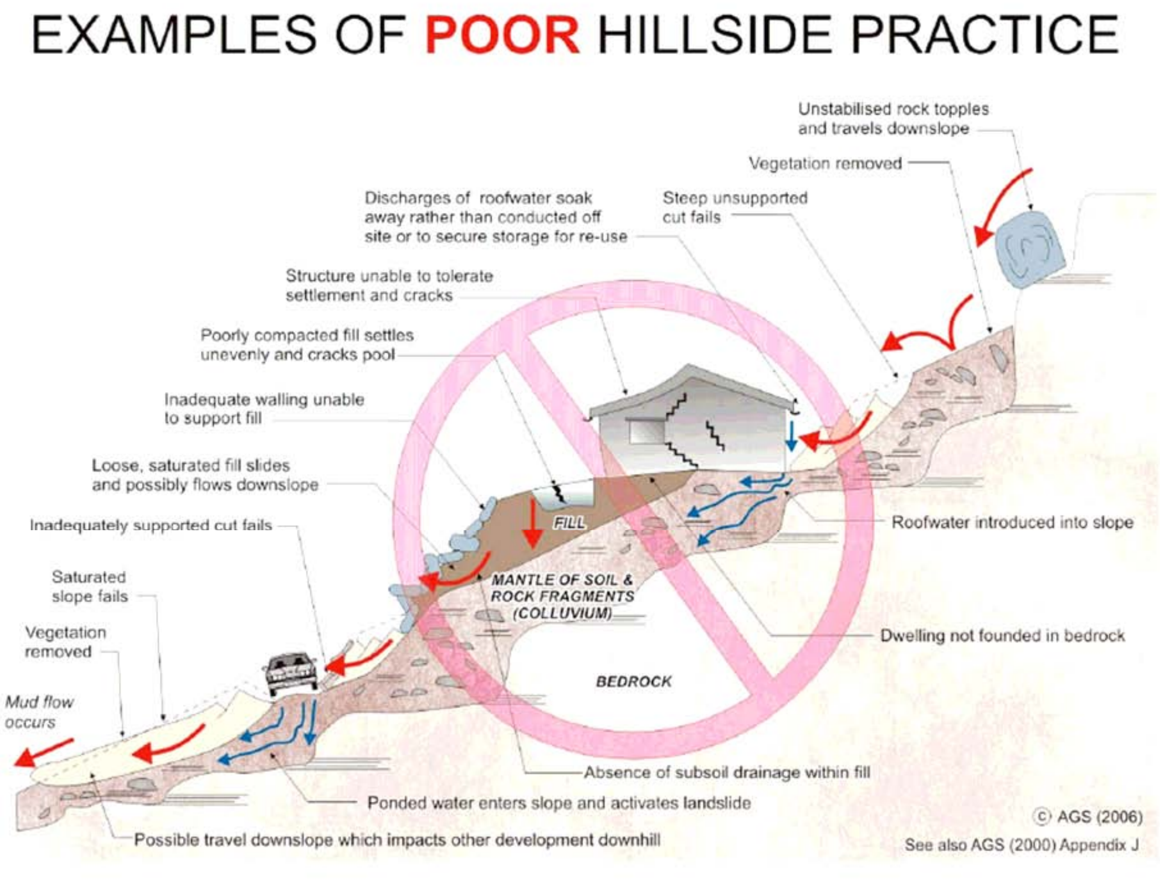
Ensure good hill slope engineering practice is adopted (examples are provided in Report Attachments). Maintain vegetation cover. Do not over-steepen existing grades without suitable shoring support. Do not place excessive load onto existing and final sloping surfaces unless designed for. Ensure appropriate foundation and footing design. . Provide / maintain appropriate surface and sub-surface drainage. Identify and control / remove existing boulders upslope of the proposed development area, as appropriate. Refer report text for further recommendations.

13 Appendix F – Hillside Construction Guidelines (AGS, 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.
PLANNING		
SITE PLANNING	Having obtained geotechnical advice, plan the development with the risk arising from the identified hazards and consequences in mind.	Plan development without regard for the Risk.
DESIGN AND CONSTRUCTION		
HOUSE DESIGN	Use flexible structures which incorporate properly designed brickwork, timber or steel frames, timber or panel cladding. Consider use of split levels. Use decks for recreational areas where appropriate.	Floor plans which require extensive cutting and filling. Movement intolerant structures.
SITE CLEARING	Retain natural vegetation wherever practicable.	Indiscriminately clear the site.
ACCESS & DRIVEWAYS	Satisfy requirements below for cuts, fills, retaining walls and drainage. Council specifications for grades may need to be modified. Driveways and parking areas may need to be fully supported on piers.	Excavate and fill for site access before geotechnical advice.
EARTHWORKS	Retain natural contours wherever possible.	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.	


EXAMPLES OF GOOD HILLSIDE PRACTICE
Vegetation retained
Surface water interception drainage
Watertight, adequately sited and founded roof water storage tanks (with due regard for impact of potential leakage)
Flexible structure
Roof water piped off site or stored
On-site detention tanks, watertight and adequately founded. Potential leakage managed by sub-soil drains
Vegetation retained
OFF STREET PARKING
ROADWAY
MANTLE OF SOIL AND ROCK FRAGMENTS (COLLUVIUM)
Pier footings into rock
Subsoil drainage may be required in slope
Cutting and filling minimised in development
Sewage effluent pumped out or connected to sewer. Tanks adequately founded and watertight. Potential leakage managed by sub-soil drains
BEDROCK
Engineered retaining walls with both surface and subsurface drainage (constructed before dwelling)
© AGS (2006)


EXAMPLES OF POOR HILLSIDE PRACTICE
Unstabilised rock topples and travels downslope
Vegetation removed
Discharges of roofwater soak away rather than conducted off site or to secure storage for re-use
Steep unsupported cut fails
Structure unable to tolerate settlement and cracks
Poorly compacted fill settles unevenly and cracks pool
Inadequate walling unable to support fill
Loose, saturated fill slides and possibly flows downslope
Inadequately supported cut fails
Saturated slope fails
Vegetation removed
Mud flow occurs
FILL
MANTLE OF SOIL & ROCK FRAGMENTS (COLLUVIUM)
BEDROCK
Roofwater introduced into slope
Dwelling not founded in bedrock
Absence of subsoil drainage within fill
Ponded water enters slope and activates landslide
Possible travel downslope which impacts other development downhill
© AGS (2006)
See also AGS (2000) Appendix J

14 Appendix G – General Geotechnical Recommendations

Geotechnical Recommendations

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These general geotechnical recommendations have been prepared by Martens to help you deliver a safe work site, to comply with your obligations, and to deliver your project. Not all are necessarily relevant to this report but are included as general reference. Any specific recommendations made in the report will override these recommendations.

Batter Slopes

Excavations in soil and extremely low to very low strength rock exceeding 0.75 m depth should be battered back at grades of no greater than 1 Vertical (V) : 2 Horizontal (H) for temporary slopes (unsupported for less than 1 month) and 1 V : 3 H for longer term unsupported slopes.

Vertical excavation may be carried out in medium or higher strength rock, where encountered, subject to inspection and confirmation by a geotechnical engineer. Long term and short term unsupported batters should be protected against erosion and rock weathering due to, for example, stormwater run-off.

Batter angles may need to be revised depending on the presence of bedding partings or adversely oriented joints in the exposed rock, and are subject to on-site inspection and confirmation by a geotechnical engineer. Unsupported excavations deeper than 1.0 m should be assessed by a geotechnical engineer for slope instability risk.

Any excavated rock faces should be inspected during construction by a geotechnical engineer to determine whether any additional support, such as rock bolts or shotcrete, is required.

Earthworks

Earthworks should be carried out following removal of any unsuitable materials and in accordance with AS3798 (2007). A qualified geotechnical engineer should inspect the condition of prepared surfaces to assess suitability as foundation for future fill placement or load application.

Earthworks inspections and compliance testing should be carried out in accordance with Sections 5 and 8 of AS3798 (2007), with testing to be carried out by a National Association of Testing Authorities (NATA) accredited testing laboratory.

Excavations

All excavation work should be completed with reference to the *Work Health and Safety (Excavation Work) Code of Practice (2015)*, by Safe Work Australia. Excavations into rock may be undertaken as follows:

1. Extremely low to low strength rock - conventional hydraulic earthmoving equipment.

2. Medium strength or stronger rock - hydraulic earthmoving equipment with rock hammer or ripping tyne attachment.

Exposed rock faces and loose boulders should be monitored to assess risk of block / boulder movement, particularly as a result of excavation vibrations.

Fill

Subject to any specific recommendations provided in this report, any fill imported to site is to comprise approved material with maximum particle size of two thirds the final layer thickness. Fill should be placed in horizontal layers of not more than 300 mm loose thickness, however, the layer thickness should be appropriate for the adopted compaction plant.

Foundations

All exposed foundations should be inspected by a geotechnical engineer prior to footing construction to confirm encountered conditions satisfy design assumptions and that the base of all excavations is free from loose or softened material and water. Water that has ponded in the base of excavations and any resultant softened material is to be removed prior to footing construction.

Footings should be constructed with minimal delay following excavation. If a delay in construction is anticipated, we recommend placing a concrete blinding layer of at least 50 mm thickness in shallow footings or mass concrete in piers / piles to protect exposed foundations.

A geotechnical engineer should confirm any design bearing capacity values, by further assessment during construction, as necessary.

Shoring - Anchors

Where there is a requirement for either soil or rock anchors, or soil nailing, and these structures penetrate past a property boundary, appropriate permission from the adjoining land owner must be obtained prior to the installation of these structures.

Shoring - Permanent

Permanent shoring techniques may be used as an alternative to temporary shoring. The design of such structures should be in accordance with the findings of this report and any further testing recommended by this report. Permanent shoring may include [but not be limited to] reinforced block work walls, contiguous and semi contiguous pile walls, secant pile walls and soldier pile walls with or without reinforced shotcrete infill panels. The choice of shoring system will depend on the type of structure, project budget and site specific geotechnical conditions.

Permanent shoring systems are to be engineer designed and backfilled with suitable granular material and free-draining drainage material. Backfill should be placed in maximum 100 mm thick layers compacted using a hand operated compactor. Care should be taken to ensure excessive compaction stresses are not transferred to retaining walls.

Shoring design should consider any surcharge loading from sloping / raised ground behind shoring structures, live loads, new structures, construction equipment, backfill compaction and static water pressures. All shoring systems shall be provided with adequate foundation designs.

Suitable drainage measures, such as geotextile enclosed 100 mm agricultural pipes embedded in free-draining gravel, should be included to redirect water that may collect behind the shoring structure to a suitable discharge point.

Shoring - Temporary

In the absence of providing acceptable excavation batters, excavations should be supported by suitably designed and installed temporary shoring / retaining structures to limit lateral deflection of excavation faces and associated ground surface settlements.

Soil Erosion Control

Removal of any soil overburden should be performed in a manner that reduces the risk of sedimentation occurring in any formal stormwater drainage system, on neighbouring land and in receiving waters. Where possible, this may be achieved by one or more of the following means:

1. Maintain vegetation where possible
2. Disturb minimal areas during excavation
3. Revegetate disturbed areas if possible

All spoil on site should be properly controlled by erosion control measures to prevent transportation of sediments off-site. Appropriate soil erosion control methods in accordance with Landcom (2004) shall be required.

Trafficability and Access

Consideration should be given to the impact of the proposed works and site subsurface conditions on trafficability within the site e.g. wet clay soils will lead to poor trafficability by tyred plant or vehicles.

Where site access is likely to be affected by any site works, construction staging should be organised such that any impacts on adequate access are minimised as best as possible.

Vibration Management

Where excavation is to be extended into medium or higher strength rock, care will be required when using a rock hammer to limit potential structural distress from excavation-induced vibrations where nearby structures may be affected by the works.

To limit vibrations, we recommend limiting rock hammer size and set frequency, and setting the hammer parallel to bedding planes and along defect planes, where possible, or as advised by a geotechnical engineer. We recommend limiting vibration peak particle velocities (PPV) caused by construction equipment or resulting from excavation at the site to 5 mm/s (AS 2187.2, 2006, Appendix J).

Waste – Spoil and Water

Soil to be disposed off-site should be classified in accordance with the relevant State Authority guidelines and requirements.

Any collected waste stormwater or groundwater should also be tested prior to discharge to ensure contaminant levels (where applicable) are appropriate for the nominated discharge location.

MA can complete the necessary classification and testing if required. Time allowance should be made for such testing in the construction program.

Water Management - Groundwater

If the proposed works are likely to intersect ephemeral or permanent groundwater levels, the management of any potential acid soil drainage should be considered. If groundwater tables are likely to be lowered, this should be further discussed with the relevant State Government Agency.

Water Management – Surface Water

All surface runoff should be diverted away from excavation areas during construction works and prevented from accumulating in areas surrounding any retaining structures, footings or the base of excavations.

Any collected surface water should be discharged into a suitable Council approved drainage system and not adversely impact downslope surface and subsurface conditions.

All site discharges should be passed through a filter material prior to release. Sump and pump methods will generally be suitable for collection and removal of accumulated surface water within any excavations.

Contingency Plan

In the event that proposed development works cause an adverse impact on geotechnical hazards, overall site stability or adjacent properties, the following actions are to be undertaken:

1. Works shall cease immediately.
2. The nature of the impact shall be documented and the reason(s) for the adverse impact investigated.
3. A qualified geotechnical engineer should be consulted to provide further advice in relation to the issue.

15 Appendix H – Notes about this Report

Information

Important Information About Your Report (1 of 2)

These notes have been prepared by Martens to help you interpret and understand the limitations of your report. Not all are necessarily relevant to all reports but are included as general reference.

Engineering Reports - Limitations

The recommendations presented in this report are based on limited investigations and include specific issues to be addressed during various phases of the project. If the recommendations presented in this report are not implemented in full, the general recommendations may become inapplicable and Martens & Associates accept no responsibility whatsoever for the performance of the works undertaken.

Occasionally, sub-surface conditions between and below the completed boreholes or other tests may be found to be different (or may be interpreted to be different) from those expected. Variation can also occur with groundwater conditions, especially after climatic changes. If such differences appear to exist, we recommend that you immediately contact Martens & Associates.

Relative ground surface levels at borehole locations may not be accurate and should be verified by on-site survey.

Engineering Reports – Project Specific Criteria

Engineering reports are prepared by qualified personnel. They are based on information obtained, on current engineering standards of interpretation and analysis, and on the basis of your unique project specific requirements as understood by Martens. Project criteria typically include the general nature of the project; its size and configuration; the location of any structures on the site; other site improvements; the presence of underground utilities; and the additional risk imposed by scope-of-service limitations imposed by the Client.

Where the report has been prepared for a specific design proposal (e.g. a three storey building), the information and interpretation may not be relevant if the design proposal is changed (e.g. to a twenty storey building). Your report should not be relied upon, if there are changes to the project, without first asking Martens to assess how factors, which changed subsequent to the date of the report, affect the report's recommendations. Martens will not accept responsibility for problems that may occur due to design changes, if not consulted.

Engineering Reports – Recommendations

Your report is based on the assumption that site conditions, as may be revealed through selective point sampling, are indicative of actual conditions throughout an area. This assumption often cannot be substantiated until project implementation has commenced. Therefore your site investigation report recommendations should only be regarded as preliminary.

Only Martens, who prepared the report, are fully familiar with the background information needed to assess whether or not the report's recommendations are valid and whether or not changes should be considered as the project develops. If another party undertakes the implementation of the recommendations of this report, there is a risk that the report will be misinterpreted and Martens cannot be held responsible for such misinterpretation.

Engineering Reports – Use for Tendering Purposes

Where information obtained from investigations is provided for tendering purposes, Martens recommend 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.

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

Engineering Reports – Data

The report as a whole presents the findings of a site assessment and should not be copied in part or altered in any way.

Logs, figures, drawings *etc* are customarily included in a Martens report and are developed by scientists, engineers or geologists based on their interpretation of field logs (assembled by field personnel), desktop studies and laboratory evaluation of field samples. These data should not under any circumstances be redrawn for inclusion in other documents or separated from the report in any way.

Engineering Reports – Other Projects

To avoid misuse of the information contained in your report it is recommended that you confer with Martens before passing your report on to another party who may not be familiar with the background and purpose of the report. Your report should not be applied to any project other than that originally specified at the time the report was issued.

Subsurface Conditions - General

Every care is taken with the report in relation to interpretation of subsurface conditions, discussion of geotechnical aspects, relevant standards and recommendations or suggestions for design and construction. However, the Company cannot always anticipate or assume responsibility for:

- Unexpected variations in ground conditions - the potential will depend partly on test point (eg. excavation or borehole) spacing and sampling frequency, which are often limited by project imposed budgetary constraints.

- Changes in guidelines, standards and policy or interpretation of guidelines, standards and policy by statutory authorities.
- The actions of contractors responding to commercial pressures.
- Actual conditions differing somewhat from those inferred to exist, because no professional, no matter how qualified, can reveal precisely what is hidden by earth, rock and time.

  The actual interface between logged materials may be far more gradual or abrupt than assumed based on the facts obtained. Nothing can be done to change the actual site conditions which exist, but steps can be taken to reduce the impact of unexpected conditions.

If these conditions occur, Martens will be pleased to assist with investigation or providing advice to resolve the matter.

Subsurface Conditions - Changes

Natural processes and the activity of man create subsurface conditions. For example, water levels can vary with time, fill may be placed on a site and pollutants may migrate with time. Reports are based on conditions which existed at the time of the subsurface exploration / assessment.

Decisions should not be based on a report whose adequacy may have been affected by time. If an extended period of time has elapsed since the report was prepared, consult Martens to be advised how time may have impacted on the project.

Subsurface Conditions - Site Anomalies

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

Report Use by Other Design Professionals

To avoid potentially costly misinterpretations when other design professionals develop their plans based on a Martens report, retain Martens to work with other project professionals affected by the report. This may involve Martens explaining the report design implications and then reviewing plans and specifications produced to see how they have incorporated the report findings.

Subsurface Conditions – Geo-environmental Issues

Your report generally does not relate to any findings, conclusions, or recommendations about the potential for hazardous or contaminated materials existing at the site unless specifically required to do so as part of Martens' proposal for works.

Specific sampling guidelines and specialist equipment, techniques and personnel are typically used to perform geo-environmental or site contamination assessments. Contamination can create major health, safety and environmental risks. If you have no information about the potential for your site to be contaminated or create an environmental hazard, you are advised to contact Martens for information relating to such matters.

Responsibility

Geo-environmental reporting relies on interpretation of factual information based on professional judgment and opinion and has an inherent level of uncertainty attached to it and is typically far less exact than the design disciplines. This has often resulted in claims being lodged against consultants, which are unfounded.

To help prevent this problem, a number of clauses have been developed for use in contracts, reports and other documents. Responsibility clauses do not transfer appropriate liabilities from Martens to other parties but are included to identify where Martens' responsibilities begin and end. Their use is intended to help all parties involved to recognise their individual responsibilities. Read all documents from Martens closely and do not hesitate to ask any questions you may have.

Site Inspections

Martens will always be pleased to provide engineering inspection services for aspects of work to which this report relates. This could range from a site visit to confirm that conditions exposed are as expected, to full time engineering presence on site. Martens is familiar with a variety of techniques and approaches that can be used to help reduce risks for all parties to a project, from design to construction.

Soil Data

Explanation of Terms (1 of 3)

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Definitions

In engineering terms, soil includes every type of uncemented or partially cemented inorganic or organic material found in the ground. In practice, if the material does not exhibit any visible rock properties and can be remoulded or disintegrated by hand in its field condition or in water, it is described as a soil. Other materials are described using rock description terms.

The methods of description and classification of soils and rocks used in this report are typically based on Australian Standard 1726 and the Unified Soil Classification System (USCS) – refer Soil Data Explanation of Terms (2 of 3). In general, descriptions cover the following properties: strength or density, colour, moisture, structure, soil or rock type and inclusions.

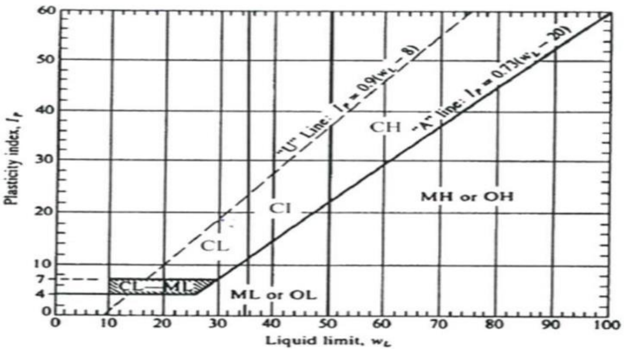
Particle Size

Soil types are described according to the predominating particle size, qualified by the grading of other particles present (e.g. sandy CLAY). Unless otherwise stated, particle size is described in accordance with the following table.

Division	Subdivision		Particle Size (mm)
Oversized	BOULDERS		>200
	COBBLES		63 to 200
Coarse Grained Soil	GRAVEL	Coarse	19 to 63
		Medium	6.7 to 19
		Fine	2.36 to 6.7
	SAND	Coarse	0.6 to 2.36
		Medium	0.21 to 0.6
		Fine	0.075 to 0.21
Fine Grained Soil	SILT		0.002 to 0.075
	CLAY		< 0.002

Plasticity Properties

Plasticity properties of cohesive soils can be assessed in the field by tactile properties or by laboratory procedures.



Soil Moisture Condition

Coarse Grained (Granular) Soil:

Dry (D):	Looks and feels dry. Cemented soils are hard, friable or powdery. Uncemented soils run freely through fingers.
Moist (M):	Feels cool and damp and is darkened in colour. Particles tend to cohere.
Wet (W):	As for moist but with free water forming on hands when handled.

Fine Grained (Cohesive) Soil:

Moist, dry of plastic limit[1] (w < PL):	Looks and feels dry. Hard, friable or powdery.
Moist, near plastic limit (w ≈ PL):	Can be moulded, feels cool and damp, is darkened in colour, at a moisture content approximately equal to the PL.
Moist, wet of plastic limit (w > PL):	Usually weakened and free water forms on hands when handled.
Wet, near liquid limit[2] (w ≈ LL)	
Wet, wet of liquid limit (w > LL)	

[1] Plastic Limit (PL): Moisture content at which soil becomes too dry to be in a plastic condition.

[2] Liquid Limit (LL): Moisture content at which soil passes from plastic to liquid state.

Consistency of Cohesive Soils

Cohesive soils refer to predominantly clay materials.
(Note: consistency is affected by soil moisture condition at time of measurement)

Term	C_u (kPa)	Field Guide
Very Soft (VS)	≤12	A finger can be pushed well into the soil with little effort. Sample exudes between fingers when squeezed in fist.
Soft (S)	>12 and ≤25	A finger can be pushed into the soil to about 25mm depth. Easily moulded by light finger pressures.
Firm (F)	>25 and ≤50	The soil can be indented about 5mm with the thumb, but not penetrated. Can be moulded by strong figure pressure.
Stiff (St)	>50 and ≤100	The surface of the soil can be indented with the thumb, but not penetrated. Cannot be moulded by fingers.
Very Stiff (VSt)	>100 and ≤200	The surface of the soil can be marked, but not indented with thumb pressure. Difficult to cut with a knife. Thumbnail can readily indent.
Hard (H)	> 200	The surface of the soil can only be marked with the thumbnail. Brittle. Tends to break into fragments.
Friable (Fr)	-	Crumbles or powders when scraped by thumbnail. Can easily be crumbled or broken into small pieces by hand.

Density of Granular Soils

Non-cohesive soils are classified on the basis of relative density, generally from standard penetration test (SPT) or Dutch cone penetrometer test (CPT) results as below:

Relative Density	%	SPT 'N' Value* (blows/300mm)	CPT Cone Value (q_c MPa)
Very loose	≤15	< 5	< 2
Loose	>15 and ≤35	5 - 10	2 - 5
Medium dense	>35 and ≤65	10 - 30	5 - 15
Dense	>65 and ≤85	30 - 50	15 - 25
Very dense	> 85	> 50	> 25

* Values may be subject to corrections for overburden pressures and equipment type and influenced by soil moisture condition at time of measurement.

Minor Components

Minor components in soils may be present and readily detectable, but have little bearing on general geotechnical classification. Terms include:

Description of components	Proportion of component in:					
	coarse grained soil				fine grained soil	
	% Fines	Terminology	% Accessory coarse fraction	Terminology	% Sand/ gravel	Terminology
Minor	≤5	Trace clay / silt, as applicable	≤15	Trace sand / gravel, as applicable	≤15	Trace sand / gravel, as applicable
	>5,≤12	With clay / silt, as applicable	>15,≤30	With sand / gravel, as applicable	>5,≤30	With sand / gravel, as applicable
Secondary	>12	Prefix soil name as 'silty' or 'clayey', as applicable	>30	Prefix soil name as 'sandy' or 'gravelly', as applicable	>30	Prefix soil name as 'sandy' or 'gravelly', as applicable

Soil Data

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Symbols for Soils and Other

SOILS	SOILS	OTHER
COBBLES/BOULDERS	SILT (ML or MH)	FILL
GRAVEL (GP or GW)	ORGANIC SILT or CLAY (OH or OL)	TALUS
Silty GRAVEL (GM)	CLAY (CL, CI or CH)	ASPHALT
Clayey GRAVEL (GC)	Silty CLAY	CONCRETE
SAND (SP or SW)	Sandy CLAY	TOPSOIL
Silty SAND (SM)	PEAT (Pt)	
Clayey SAND (SC)	Gravelly CLAY	

Unified Soil Classification Scheme (USCS)

FIELD IDENTIFICATION PROCEDURES (Excluding particles larger than 63 mm and basing fractions on estimated mass)

				FIELD IDENTIFICATION PROCEDURES	USCS	Primary Name
COARSE GRAINED SOILS More than 65 % of material less than 63 mm is larger than 0.075 mm	(A 0.075 mm particle is about the smallest particle visible to the naked eye)	GRAVELS More than half of coarse fraction is larger than 2.36 mm.	GRAVEL and GRAVEL-SAND Mixtures (≤ 5% fines)	Wide range in grain size and substantial amounts of all intermediate particle sizes; not enough fines to bind coarse grains; no dry strength	GW	GRAVEL
				Predominantly one size or a range of sizes with some intermediate sizes missing; not enough fines to bind coarse grains; no dry strength	GP	GRAVEL
			GRAVEL-SILT and GRAVEL-SAND-SILT mixtures (≥12% fines) [1]	With excess non-plastic fines (for identification procedures see ML below); zero to medium dry strength; may also contain sand	GM	Silty GRAVEL
				With excess plastic fines (for identification procedures see CL below); medium to high dry strength; may also contain sand	GC	Clayey GRAVEL
		SANDS More than half of coarse fraction is smaller than 2.36 mm	SAND and GRAVEL-SAND mixtures (≤5% fines)	Wide range in grain sizes and substantial amounts of all intermediate sizes; not enough fines to bind coarse grains; no dry strength.	SW	SAND
				Predominantly one size or a range of sizes with some intermediate sizes missing; not enough fines to bind coarse grains; no dry strength	SP	SAND
			SAND-SILT and SAND-CLAY mixtures (≥12% fines) [1]	With excess non-plastic fines (for identification procedures see ML below); zero to medium dry strength;	SM	Silty SAND
				With excess plastic fines (for identification procedures see CL below); medium to high dry strength	SC	Clayey SAND

IDENTIFICATION PROCEDURES ON FRACTIONS < 0.2 MM

	DRY STRENGTH (Crushing Characteristics)	DILATANCY	TOUGHNESS	DESCRIPTION	USCS	Primary Name
FINE GRAINED SOILS More than 35 % of material less than 63 mm is smaller than 0.075 mm	None to Low	Quick to Slow	Low	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands or silt with low plasticity [2]	ML	SILT [3]
	Medium to High	None to Slow	Medium	Inorganic clays of low to medium plasticity, gravely clays, sandy clays, silty clays, lean clays	CL (or CI[4])	CLAY
	Low to Medium	Slow	Low	Organic slits and organic silty clays of low plasticity	OL	Organic SILT or CLAY
	Low to Medium	None to Slow	Low to Medium	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts	MH	SILT [3]
	High to Very High	None	High	Inorganic clays of high plasticity, fat clays	CH	CLAY
	Medium to High	None to Very Slow	Low to Medium	Organic clays of medium to high plasticity, organic silt of high plasticity	OH	Organic SILT or CLAY
HIGHLY ORGANIC SOILS	Readily identified by colour, odour, spongy feel and frequently by fibrous texture				Pt	PEAT

Notes:

1. Between 5% and 12% - dual classification, e.g. GP-GM.
2. Low Plasticity Clay – Liquid Limit $W_L \leq 35\%$; Medium Plasticity Clay – Liquid limit $W_L > 35\%$, $\leq 50\%$; High Plasticity Clay - Liquid limit $W_L > 50\%$.
3. Low Plasticity Silt – Liquid Limit $W_L \leq 50\%$; High Plasticity Silt - Liquid limit $W_L > 50\%$.
4. CI may be adopted for clay of medium plasticity to distinguish from clay of low plasticity.

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Soil Agricultural Classification Scheme

In some situations, such as where soils are to be used for effluent disposal purposes, soils are often more appropriately classified in terms of traditional agricultural classification schemes. Where a Martens report provides agricultural classifications, these are undertaken in accordance with descriptions by Northcote, K.H. (1979) *The factual key for the recognition of Australian Soils*, Rellim Technical Publications, NSW, p 26 - 28.

Symbol	Field Texture Grade	Behaviour of moist bolus	Ribbon length	Clay content (%)
S	Sand	Coherence nil to very slight; cannot be moulded; single grains adhere to fingers	0 mm	< 5
LS	Loamy sand	Slight coherence; discolours fingers with dark organic stain	6.35 mm	5
CLS	Clayey sand	Slight coherence; sticky when wet; many sand grains stick to fingers; discolours fingers with clay stain	6.35mm - 1.3cm	5 - 10
SL	Sandy loam	Bolus just coherent but very sandy to touch; dominant sand grains are of medium size and are readily visible	1.3 - 2.5	10 - 15
FSL	Fine sandy loam	Bolus coherent; fine sand can be felt and heard	1.3 - 2.5	10 - 20
SCL^{-}	Light sandy clay loam	Bolus strongly coherent but sandy to touch, sand grains dominantly medium size and easily visible	2.0	15 - 20
L	Loam	Bolus coherent and rather spongy; smooth feel when manipulated but no obvious sandiness or silkiness; may be somewhat greasy to the touch if much organic matter present	2.5	25
Lfsy	Loam, fine sandy	Bolus coherent and slightly spongy; fine sand can be felt and heard when manipulated	2.5	25
SiL	Silt loam	Coherent bolus, very smooth to silky when manipulated	2.5	25 + > 25 silt
SCL	Sandy clay loam	Strongly coherent bolus sandy to touch; medium size sand grains visible in a finer matrix	2.5 - 3.8	20 - 30
CL	Clay loam	Coherent plastic bolus; smooth to manipulate	3.8 - 5.0	30 - 35
SiCL	Silty clay loam	Coherent smooth bolus; plastic and silky to touch	3.8 - 5.0	30- 35 + > 25 silt
FSCL	Fine sandy clay loam	Coherent bolus; fine sand can be felt and heard	3.8 - 5.0	30 - 35
SC	Sandy clay	Plastic bolus; fine to medium sized sands can be seen, felt or heard in a clayey matrix	5.0 - 7.5	35 - 40
SiC	Silty clay	Plastic bolus; smooth and silky	5.0 - 7.5	35 - 40 + > 25 silt
LC	Light clay	Plastic bolus; smooth to touch; slight resistance to shearing	5.0 - 7.5	35 - 40
LMC	Light medium clay	Plastic bolus; smooth to touch, slightly greater resistance to shearing than LC	7.5	40 - 45
MC	Medium clay	Smooth plastic bolus, handles like plasticine and can be moulded into rods without fracture, some resistance to shearing	> 7.5	45 - 55
HC	Heavy clay	Smooth plastic bolus; handles like stiff plasticine; can be moulded into rods without fracture; firm resistance to shearing	> 7.5	> 50

Rock Data

Symbols for Rock

SEDIMENTARY ROCK

BRECCIA

CONGLOMERATE

CONGLOMERATIC SANDSTONE

SANDSTONE/QUARTZITE

SILTSTONE

MUDSTONE/CLAYSTONE

SHALE

COAL

LIMESTONE

LITHIC TUFF

IGNEOUS ROCK

GRANITE

DOLERITE/BASALT

METAMORPHIC ROCK

SLATE, PHYLLITE, SCHIST

GNEISS

METASANDSTONE

METASILTSTONE

METAMUDSTONE

Definitions

Descriptive terms used for Rock by Martens are based on AS1726 and encompass rock substance, defects and mass.

Rock Material	The intact rock that is bounded by defects.
Rock Defect	Discontinuity, fracture, break or void in the material or minerals across which there is little or no tensile strength.
Rock Structure	The nature and configuration of the different defects within the rock mass and their relationship to each other.
Rock Mass	The entirety of the system formed by all of the rock material and all of the defects that are present.

Degree of Weathering

Rock weathering is defined as the degree of decline in rock structure and grain property and can be determined in the field.

Term	Symbol	Definition
Residual soil[1]	RS	Material is weathered to such an extent that it has soil properties. Mass structure, material texture, and fabric of original rock are no longer visible, but the soil has not been significantly transported.
Extremely weathered[1]	XW	Material is weathered to such an extent that it has soil properties - i.e. it can be remoulded and can be classified according to the Unified Classification System. Mass structure and material texture and fabric of original rock are still visible.
Highly weathered[2]	HW	The whole of the rock material is discoloured, usually by iron staining or bleaching to the extent that the original colour of the rock is not recognisable. Rock strength is significantly changed by weathering. Some primary minerals have weathered to clay minerals. Porosity may be increased by leaching, or may be decreased due to deposition of weathering products in pores.
Moderately weathered[2]	MW	The whole of the rock material is discoloured, usually by iron staining or bleaching to the extent that the colour of the rock is not recognisable. Rock strength shows little or no change from fresh rock.
Slightly weathered	SW	Rock is partially discoloured with staining or bleaching along joints but shows little or no change of strength from fresh rock.
Fresh	FR	Rock substance unaffected by weathering. No sign of decomposition of individual materials or colour changes.

Notes:

1 RS and EW material is described using soil descriptive terms.

2. The term "Distinctly Weathered" (DW) may be used to cover the range of substance weathering between EW and SW

Rock Strength

Rock strength is defined by the Point Load Strength Index (Is 50) and refers to the strength of the rock substance in the direction normal to the loading. The test procedure is described by the International Society of Rock Mechanics.

Term (Strength)	I_s (50) MPa	Uniaxial Compressive Strength MPa	Field Guide	Symbol
Very low	>0.03 ≤0.1	0.6 – 2	May be crumbled in the hand. Sandstone is 'sugary' and friable.	VL
Low	>0.1 ≤0.3	2 – 6	Core 150mm long x 50mm diameter may be broken by hand and easily scored with a knife. Sharp edges of core may be friable and break during handling.	L
Medium	>0.3 ≤1.0	6 – 20	Core 150mm long x 50mm diameter can be broken by hand with considerable difficulty. Readily scored with a knife.	M
High	>1 ≤3	20 – 60	Core 150mm long x 50mm diameter cannot be broken by unaided hands, can be slightly scratched or scored with a knife. Breaks with single blow from pick.	H
Very high	>3 ≤10	60 – 200	Core 150mm long x 50mm diameter, broken readily with hand held hammer. Cannot be scratched with knife. Breaks after more than one pick strike.	VH
Extremely high	>10	>200	A piece of core 150mm long x 50mm diameter is difficult to break with hand held hammer. Rings when struck with a hammer.	EH

Degree of Fracturing

This classification applies to diamond drill cores and refers to the spacing of all types of natural fractures along which the core is discontinuous. These include bedding plane partings, joints and other rock defects, but exclude fractures such as drilling breaks (DB) or handling breaks (HB).

Term	**Description**
Fragmented	The core is comprised primarily of fragments of length less than 20 mm, and mostly of width less than core diameter.
Highly fractured	Core lengths are generally less than 20 mm to 40 mm with occasional fragments.
Fractured	Core lengths are mainly 30 mm to 100 mm with occasional shorter and longer sections.
Slightly fractured	Core lengths are generally 300 mm to 1000 mm, with occasional longer sections and sections of 100 mm to 300 mm.
Unbroken	The core does not contain any fractures.

Rock Core Recovery

TCR = Total Core Recovery

$$= \frac{\text{Length of core recovered}}{\text{Length of core run}} \times 100\,\%$$

SCR = Solid Core Recovery

$$= \frac{\sum \text{Length of cylindrical core recovered}}{\text{Length of core run}} \times 100\,\%$$

RQD = Rock Quality Designation

$$= \frac{\sum \text{Axial lengths of core} > 100\text{ mm long}}{\text{Length of core run}} \times 100\,\%$$

Rock Strength Tests

▼ Point load strength Index (Is50) - axial test (MPa)

▶ Point load strength Index (Is50) - diametral test (MPa)

● Uniaxial compressive strength (UCS) (MPa)

Defect Type Abbreviations and Descriptions

Defect Type (with inclination given)

BP	Bedding plane parting
FL	Foliation
CL	Cleavage
JT	Joint
FC	Fracture
SZ/SS	Sheared zone/ seam (Fault)
CZ/CS	Crushed zone/ seam
DZ/DS	Decomposed zone/ seam
FZ	Fractured Zone
IS	Infilled seam
VN	Vein
CO	Contact
HB	Handling break
DB	Drilling break

Planarity

Pl	Planar
Cu	Curved
Un	Undulating
St	Stepped
Ir	Irregular
Dis	Discontinuous

Roughness

Pol	Polished
Sl	Slickensided
Sm	Smooth
Ro	Rough
VR	Very rough

Thickness

Zone	> 100 mm
Seam	> 2 mm < 100 mm
Plane	< 2 mm

Coating or Filling

Cn	Clean
Sn	Stain
Ct	Coating
Vnr	Veneer
Fe	Iron Oxide
X	Carbonaceous
Qz	Quartzite
MU	Unidentified mineral

Inclination

Inclination of defect is measured from perpendicular to and down the core axis.
Direction of defect is measured clockwise (looking down core) from magnetic north.

Test, Drill and Excavation Methods

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Sampling

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

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

Undisturbed samples may be taken by pushing a thin-walled sampling tube, e.g. U_{50} (50 mm internal diameter thin walled tube), into soils and withdrawing a soil sample 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 effective only in cohesive soils. Other sampling methods may be used. Details of the type and method of sampling are given in the report.

Drilling / Excavation Methods

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

Hand Excavation - in some situations, excavation using hand tools, such as mattock and spade, may be required due to limited site access or shallow soil profiles.

Hand Auger - the hole is advanced by pushing and rotating either a sand or clay auger, generally 75-100 mm in diameter, into the ground. The penetration depth is usually limited to the length of the auger pole; however extender pieces can be added to lengthen this.

Test Pits - these are excavated with a backhoe or a tracked excavator, allowing close examination of the in-situ soils and, if it is safe to descend into the pit, collection of bulk disturbed samples. The depth of penetration is limited to about 3 m for a backhoe and up to 6 m for an excavator. A potential disadvantage is the disturbance caused by the excavation.

Large Diameter Auger (e.g. Pengo) - the hole is advanced by a rotating plate or short spiral auger, generally 300 mm or larger in diameter. The cuttings are returned to the surface at intervals (generally of not more than 0.5 m) 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 sampling.

Continuous Sample Drilling (Push Tube) - the hole is advanced by pushing a 50 - 100 mm diameter socket into the ground and withdrawing it at intervals to extrude the sample. This is the most reliable method of drilling in soils, since moisture content is unchanged and soil structure, strength *etc.* is only marginally affected.

Continuous Spiral Flight Augers - the hole is advanced using 90 - 115 mm 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 in sands above the water table. Samples are returned to the surface or, or may be collected after withdrawal of the auger flights, but they are very disturbed and may be contaminated. Information from the drilling (as distinct from specific sampling by SPTs or undisturbed samples) is of relatively lower reliability, due to remoulding, contamination or softening of samples by ground water.

Non-core Rotary Drilling - the hole is advanced by a rotary bit, with water being pumped down the drill rods and returned up the annulus, carrying the drill cuttings. Only major changes in stratification can be determined from the cuttings, together with some information from 'feel' and rate of penetration.

Rotary Mud Drilling - similar to rotary drilling, but using drilling mud as a circulating fluid. The mud tends to mask the cuttings and reliable identification is again only possible from separate intact sampling (eg. from SPT).

Continuous Core Drilling - a continuous core sample is obtained using a diamond tipped core barrel of usually 50 mm internal diameter. Provided full core recovery is achieved (not always possible in very weak or fractured rocks and granular soils), this technique provides a very reliable (but relatively expensive) method of investigation.

In-situ Testing and Interpretation

Cone Penetrometer Testing (CPT)
Cone penetrometer testing (sometimes referred to as Dutch Cone) described in this report has been carried out using an electrical friction cone penetrometer.

The test is described in AS 1289.6.5.1-1999 (R2013). In the test, a 35 mm diameter rod with a cone tipped end is pushed continuously into the soil, the reaction being provided by a specially designed truck or rig which is fitted with an hydraulic ram system.

Measurements are made of the end bearing resistance on the cone and the friction resistance on a separate 130 mm long sleeve, immediately behind the cone. Transducers in the tip of the assembly are connected by electrical wires passing through the push rod centre to an amplifier and recorder unit mounted on the control truck. As penetration occurs (at a rate of approximately 20 mm per second) the information is output on continuous chart recorders. The plotted results given in this report have been traced from the original records. The information provided on the charts comprises:

(i) Cone resistance (q_c) - the actual end bearing force divided by the cross sectional area of the cone, expressed in MPa.

(ii) Sleeve friction (q_f) - the frictional force of the sleeve divided by the surface area, expressed in kPa.

(iii) Friction ratio - the ratio of sleeve friction to cone resistance, expressed in percent.

There are two scales available for measurement of cone resistance. The lower (A) scale (0 - 5 MPa) is used in very soft soils where increased sensitivity is required and is shown in the graphs as a dotted line. The main (B) scale (0 - 50 MPa) is less sensitive and is shown as a full line.

The ratios of the sleeve resistance to cone resistance will vary with the type of soil encountered, with higher relative friction in clays than in sands. Friction ratios of 1 % - 2 % are commonly encountered in sands and very soft clays rising to 4 % - 10 % in stiff clays.

In sands, the relationship between cone resistance and SPT value is commonly in the range:

$$q_c \text{ (MPa)} = (0.4 \text{ to } 0.6)\ N \text{ (blows/300 mm)}$$

In clays, the relationship between undrained shear strength and cone resistance is commonly in the range:

$$q_c = (12 \text{ to } 18)\ C_u$$

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

Inferred stratification as shown on the attached reports is assessed from the cone and friction traces and from experience and information from nearby boreholes *etc.* This information is presented for general guidance, but must be regarded as being to some extent interpretive. The test method provides a continuous profile of engineering properties, and where precise information on soil classification is required, direct drilling and sampling may be preferable.

Standard Penetration Testing (SPT)

Standard penetration tests are used mainly in non-cohesive soils, but occasionally also in cohesive soils as a means of determining density or strength and also of obtaining a relatively undisturbed sample.

The test procedure is described in AS 1289.6.3.1-2004. The test is carried out in a borehole by driving a 50 mm diameter split sample tube 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 penetration depth increments and the 'N' value is taken as the number of blows for the last two 150 mm depth increments (300 mm total penetration). In dense sands, very hard clays or weak rock, the full 450 mm penetration may not be practicable and the test is discontinued. The test results are reported in the following form:

(i) Where full 450 mm penetration is obtained with successive blow counts for each 150 mm of say 4, 6 and 7 blows:

as 4, 6, 7
N = 13

(ii) Where the test is discontinued, short of full penetration, say after 15 blows for the first 150mm and 30 blows for the next 40mm

as 15, 30/40 mm.

The results of the tests can be related empirically to the engineering properties of the soil. Occasionally, the test method is used to obtain samples in 50 mm diameter thin walled sample tubes in clays. In such circumstances, the test results are shown on the borehole logs in brackets.

Dynamic Cone (Hand) Penetrometers

Hand penetrometer tests are carried out by driving a rod into the ground with a falling weight hammer and measuring the blows for successive 150mm increments of penetration. Normally, there is a depth limitation of 1.2m but this may be extended in certain conditions by the use of extension rods. Two relatively similar tests are used.

Perth sand penetrometer (PSP) - a 16 mm diameter flat ended rod is driven with a 9 kg hammer, dropping 600 mm. The test, described in AS 1289.6.3.3-1997 (R2013), was developed for testing the density of sands (originating in Perth) and is mainly used in granular soils and filling.

Cone penetrometer (DCP) - sometimes known as the Scala Penetrometer, a 16 mm rod with a 20 mm diameter cone end is driven with a 9 kg hammer dropping 510 mm. The test, described in AS 1289.6.3.2-1997 (R2013), was developed initially for pavement sub-grade investigations, with correlations of the test results with California Bearing Ratio published by various Road Authorities.

Pocket Penetrometers

The pocket (hand) penetrometer (PP) is typically a light weight spring hand operated device with a stainless steel loading piston, used to estimate unconfined compressive strength, q_u, (UCS in kPa) of a fine grained soil in field conditions. In use, the free end of the piston is pressed into the soil at a uniform penetration rate until a line, engraved near the piston tip, reaches the soil surface level. The reading is taken from a gradation scale, which is attached to the piston via a built-in spring mechanism and calibrated to kilograms per square centimetre (kPa) UCS. The UCS measurements are used to evaluate consistency of the soil in the field moisture condition. The results may be used to assess the undrained shear strength, C_u, of fine grained soil using the approximate relationship:

$q_u = 2 \times C_u$.

It should be noted that accuracy of the results may be influenced by condition variations at selected test surfaces. Also, the readings obtained from the PP test are based on a small area of penetration and could give misleading results. They should not replace laboratory test results. The use of the results from this test is typically limited to an assessment of consistency of the soil in the field and not used directly for design of foundations.

Test Pit / Borehole Logs

Test pit / borehole log(s) presented herein are an engineering and / or geological interpretation of the subsurface conditions. Their reliability will depend to some extent on frequency of sampling and methods of excavation / drilling. Ideally, continuous undisturbed sampling or excavation / core drilling will provide the most reliable assessment but this is not always practicable, or possible to justify on economic grounds. In any case, the test pit / borehole logs represent only a very small sample of the total subsurface profile.

Interpretation of the information and its application to design and construction should therefore take into account the spacing of test pits / boreholes, the frequency of sampling and the possibility of other than 'straight line' variation between the test pits / boreholes.

Laboratory Testing

Laboratory testing is carried out in accordance with AS 1289 Methods of Testing Soil for Engineering Purposes. Details of the test procedure used are given on the individual report forms.

Ground Water

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

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

More reliable measurements can be made by installing standpipes, which are read at intervals over several days, or 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 a perched water table.

Test, Drill and Excavation Methods

Explanation of Terms (3 of 3)

DRILLING / EXCAVATION METHOD

HA	Hand Auger	RD	Rotary Blade or Drag Bit	NQ	Diamond Core - 47 mm
AD/V	Auger Drilling with V-bit	RT	Rotary Tricone bit	NMLC	Diamond Core – 51.9 mm
AD/T	Auger Drilling with TC-Bit	RAB	Rotary Air Blast	HQ	Diamond Core – 63.5 mm
AS	Auger Screwing	RC	Reverse Circulation	HMLC	Diamond Core – 63.5 mm
HSA	Hollow Stem Auger	CT	Cable Tool Rig	DT	Diatube Coring
S	Excavated by Hand Spade	PT	Push Tube	NDD	Non-destructive digging
BH	Tractor Mounted Backhoe	PC	Percussion	PQ	Diamond Core - 83 mm
JET	Jetting	E	Tracked Hydraulic Excavator	X	Existing Excavation

SUPPORT

Nil	No support	S	Shotcrete	RB	Rock Bolt
C	Casing	Sh	Shoring	SN	Soil Nail
WB	Wash bore with Blade or Bailer	WR	Wash bore with Roller	T	Timbering

WATER

▽ Water level at date shown

▷ Water inflow

◁ Partial water loss

◀ Complete water loss

GROUNDWATER NOT OBSERVED (NO) — The observation of groundwater, whether present or not, was not possible due to drilling water, surface seepage or cave in of the borehole/test pit.

GROUNDWATER NOT ENCOUNTERED (NX) — The borehole/test pit was dry soon after excavation. However, groundwater could be present in less permeable strata. Inflow may have been observed had the borehole/test pit been left open for a longer period.

PENETRATION / EXCAVATION RESISTANCE

- L Low resistance: Rapid penetration possible with little effort from the equipment used.
- M Medium resistance: Excavation possible at an acceptable rate with moderate effort from the equipment used.
- H High resistance: Further penetration possible at slow rate & requires significant effort equipment.
- R Refusal/ Practical Refusal. No further progress possible without risk of damage/ unacceptable wear to digging implement / machine.

These assessments are subjective and dependent on many factors, including equipment power, weight, condition of excavation or drilling tools, and operator experience.

SAMPLING

D	Small disturbed sample	W	Water Sample	C	Core sample
B	Bulk disturbed sample	G	Gas Sample	CONC	Concrete Core

U63 Thin walled tube sample - number indicates nominal undisturbed sample diameter in millimetres

TESTING

SPT	Standard Penetration Test to AS1289.6.3.1-2004
4,7,11	4,7,11 = Blows per 150mm.
N=18	'N' = Recorded blows per 300mm penetration following 150mm seating
DCP	Dynamic Cone Penetration test to AS1289.6.3.2-1997. 'n' = Recorded blows per 150mm penetration

Notes:

RW	Penetration occurred under rod weight only
HW	Penetration occurred under hammer and rod weight only
20/100mm	Where practical refusal or hammer double bouncing occurred, blows and penetration for that interval are reported (e.g. 20 blows for 100 mm penetration)

CPT	Static cone penetration test
CPTu	CPT with pore pressure (u) measurement
PP	Pocket penetrometer test expressed as instrument reading (kPa)
FP	Field permeability test over section noted
VS	Field vane shear test expressed as uncorrected shear strength (sv = peak value, sr = residual value)
PM	Pressuremeter test over section noted
PID	Photoionisation Detector reading in ppm
WPT	Water pressure tests

SOIL DESCRIPTION

Density		Consistency		Moisture	
VL	Very loose	VS	Very soft	D	Dry
L	Loose	S	Soft	M	Moist
MD	Medium dense	F	Firm	W	Wet
D	Dense	St	Stiff	Wp	Plastic limit
VD	Very dense	VSt	Very stiff	Wl	Liquid limit
		H	Hard		

ROCK DESCRIPTION

Strength		Weathering	
VL	Very low	EW	Extremely weathered
L	Low	HW	Highly weathered
M	Medium	MW	Moderately weathered
H	High	SW	Slightly weathered
VH	Very high	FR	Fresh
EH	Extremely high		