Joanna Ward

martens consulting engineers

Preliminary Geotechnical Assessment: 10 Daisy Street, North Balgowlah, NSW.

ENVIRONMENTAL

WATER

WASTEWATER

GEOTECHNICAL

CIVIL

PROJECT MANAGEMENT

P2209086JR01V01
August 2022

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All enquiries regarding this project are to be directed to the Project Manager.

Contents

1 PROPOSED DEVELOPMENT AND INVESTIGATION SCOPE 5

2 GENERAL SITE DETAILS AND SUBSURFACE CONDITIONS 6

2.1 General Site Details 6

2.2 Subsurface Conditions 7

2.3 Groundwater 7

3 GEOTECHNICAL ASSESSMENT 8

3.1 Preliminary Material Properties 8

3.2 Risk of Slope Instability 9

4 GEOTECHNICAL RECOMMENDATION 10

4.1 Excavatability 10

4.2 Excavation Support 10

4.3 Retaining Structures 11

4.4 Earth Pressure Coefficients 11

4.5 Foundations 11

Swimming Pool 11

4.6 Drainage Requirements 12

4.7 Site Classification 12

5 PROPOSED ADDITIONAL WORKS 13

5.1 Work Prior to Construction Certificate 13

5.2 Construction Monitoring and Inspection 13

6 REFERENCES 14

Attachments

ATTACHMENT A – GEOTECHNICAL TESTING PLAN

ATTACHMENT B – BOREHOLE LOGS

ATTACHMENT C – DCP ‘N’ COUNTS

ATTACHMENT D – GENERAL GEOTECHNICAL RECOMMENDATIONS

ATTACHMENT E – NOTES ABOUT THIS REPORT

1 Proposed Development and Investigation Scope

Table 1 summarises proposed development details and investigation scope.

Table 1: Summary of proposed development details and investigation scope.

Item	Details
Property Address	10 Daisy Street, North Balgowlah, NSW 2093 ('the site').
Legal Identifier	Lot 9 in DP 8443 (CMS, 2022).
Site Area	Approximately 958 m^2 (IC, 2022).
LGA	Northern Beaches Council ('Council').
Proposed Development	We understand from the architectural plans (LPL, 2022) and client provided information that the alterations and additions comprise: o A new (5.0 x 3.0 m) balcony with tile roof extension towards the western portion of the existing two storey dwelling. o A new swimming pool and timber deck with a finished floor level (FFL) of approximately 44.17 mAHD and 45.92 mAHD, respectively. The swimming pool will extend along the northern portion of the site and adjacent to new balcony. o New timber / masonry retaining walls along the northern and southern property boundaries. o Internal masonry retaining walls with timber picket fencing adjacent to the proposed swimming pool. o New timber walkway along the southwestern portion of the site. o Associated landscaping across the site, including the installation of additional timber paling fences. It is understood that the proposed development will require excavations up to approximately 1 - 2 m for the proposed swimming pool and timber deck.
Assessment Purpose	Geotechnical assessment to support a Development Application (DA) and to assist the structural design for the proposed swimming pool and dwelling additions.
Investigation Scope of Work	Field investigations conducted on 26 July 2022 included: o Review of DBYD survey plans. o General site walkover to review site topography, drainage and geology. o Drilling of three boreholes (BH101 to BH103) using hand auger to a maximum depth of approximately 1.25 metres below ground level (mbgl). o Six Dynamic Cone Penetrometer (DCP) tests (DCP101a, DCP101b, DCP102 to DCP105) adjacent to each borehole location to assess soil consistency up to 1.08 mbgl. o Collection of soil samples for future reference. Investigation locations are shown in Attachment A.

2 General Site Details and Subsurface Conditions

2.1 General Site Details

General site details based on desktop review, site walkover and site investigation findings are summarised in Table 2.

Table 2: Summary of general site details and subsurface conditions.

Item	Comment
Topography	Within gently undulating to sloping terrain, near the base of a northwest to southeast aligned ridge. The site is located approximately 245 m northwest of Burnt Bridge Creek with local reliefs between 20 - 120 m.
Typical slopes and aspect	The site generally has a south easterly aspect with overall grades of approximately less than 10 %.
Site Elevation	Site elevation decreases from approximately 50.95 mAHD in the west to 42.91 mAHD in the east (CMS, 2022).
Expected Geology	Hawkesbury Sandstone (Rh) comprising medium to coarse grained quartz sandstone, very minor shale and laminite lenses (Herbert, 1983).
Soil Landscape	The NSW Office of Environment and Heritage's (OEH) information system (eSPADE) indicates the site as being part of the Lambert (la) soil landscape, consisting of shallow (<50 cm) sandy clay loams / sandy loams on crests and benches grading to shallow and moderately deep (<150 cm) clayey sands over Hawkesbury sandstone. The soil landscape is often associated with very high soil erosion hazard, seasonally perched water tables, and shallow but highly permeable soils.
Existing Development	The existing site development includes: o An existing two storey residential dwelling towards the north eastern portion of the site. o A single storey granny flat to western portion of the site, with a garage and concrete driveway leading to Daisy Street. o A concrete patio and lawn lying immediately west of the existing two storey dwelling. An existing garden lawn also lies immediately above an excavated rock face behind the existing granny flat to the west.
Neighbouring environment	The site is bordered by: o Daisy Steet to the east. o Residential developments adjacent to the western, northern and southern site boundaries.
Vegetation	Grass and minor trees towards the rear property boundary.
Drainage	Via overland flow towards the southeast into Council stormwater network and towards Lurline Bay.

2.2 Subsurface Conditions

Investigation revealed the following generalised subsurface units likely underlie the site below ground surface level:

Unit A: Fill comprising silty sand and mixed gravels encountered up to 0.6 mbgl (BH103) to the western portion of the site. The fill is expected to have been placed under uncontrolled conditions for site levelling purposes.

Unit B: Residual soils comprising:

Unit B1: Stiff sandy clays encountered up to approximately 0.7 mbgl (BH102).

Unit B2: Medium dense to dense clayey sands encountered up to approximately 1.25 mbgl (BH102).

Based on DCP refusal depths, it is inferred that the residual soil profile may be underlain by weathered sandstone bedrock between approximately 0.2 mbgl to 1.25 mbgl.

Encountered conditions are described in more detail on borehole logs in Attachment B and associated explanatory notes in Attachment E. For DCP test results refer to Attachment C.

2.3 Groundwater

Groundwater inflow was encountered during borehole drilling at 0.5 mbgl (BH103) and 1.1 mbgl (BH102). Given the site aspect and topography of the site we anticipate that ephemeral perched groundwater was encountered within the soil originating from infiltration of surface water during recent rainfall events.

3 Geotechnical Assessment

3.1 Preliminary Material Properties

Preliminary material properties, inferred from observations during borehole drilling, such as auger penetration resistance, DCP test results as well as engineering judgement are summarised in Table 3.

Table 3: Preliminary material properties.

Units	Layer [1]	$\gamma_{in\text{-}situ}$ (kN/3) [2]	Cu (kPa) [3]	C' (kPa) [4]	Ø' (deg) [5]	E' (MPa) [6]
A	FILL (uncontrolled): MIXED GRAVELS / Silty SAND	17	NA [7]	0	30	5
B1	RESIDUAL: Sandy CLAY (stiff)	20	40	1	26	7
B2	RESIDUAL: Clayey SAND (medium dense to dense)	19	NA [7]	0	34	12
	WEATHERED ROCK: SANDSTONE (inferred highly weathered, very low to low strength)	22	NA [7]	30	28	70

Notes:

1. Refer to borehole logs in Attachment B for material description details.
2. Inferred material in-situ unit weight, based on visual assessment and DCP testing.
3. Average undrained shear strength estimate assuming normally consolidated clay.
4. Effective cohesion.
5. Average effective internal friction angle estimate assuming drained conditions.
6. Average effective elastic modulus estimate.
7. Not applicable.

3.2 Risk of Slope Instability

A preliminary walkover of the site was undertaken to identify any potentially unstable areas. Site inspection revealed that no evidence of former large scale land instability was observed within the site and surrounding land. Observations of the excavated rock face at the rear of the site revealed the following:

- The rock face has a height of approximately 1.7 m and extends in a north – south alignment from the rear of the granny flat. The rock face comprised moderately to slightly weathered, inferred low strength sandstone with larger weathered areas extending towards the south eastern boundary.

- Possible jointing along weathered zones of the rock face and overhanging were identified.

Based on our preliminary observations, the rock face appears to be in a generally stable condition, however, we recommend that any loose boulders be removed and an inspection be carried out prior to construction, by a geotechnical engineer to assess any required stabilisation measures (if required). A detailed slope risk assessment in accordance with AGS (2007) guidelines was not undertaken.

4 Geotechnical Recommendation

The following recommendations are provided for the proposed development. Further general geotechnical recommendations are provided in Attachment D.

4.1 Excavatability

Excavations for the proposed swimming pool and retaining wall is expected to encounter inferred highly weathered, very low to low strength sandstone bedrock below existing soils. Both soils and highly weathered very low strength bedrock should be readily excavated using conventional earthmoving equipment with low to medium strength bands requiring localised use of rock breaking equipment or ripping tyne.

Both noise and vibration will be generated by the proposed excavation work within these bedrock materials. Vibration control and management will be required in accordance with AS2187.2 (2006).

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

4.2 Excavation Support

Where there is sufficient setback between the excavation and site boundary, a batter slope of 1V:1.5H may be adopted in fill and residual soils and 0.75H:1V in the bedrock, subject to inspection and approval by an experienced geotechnical engineer to confirm adopted batter slopes and to assess any impact on adjacent structures or infrastructure.

Where there is insufficient setback or where adjacent foundations / infrastructure are present within 1.5 m of batter slope crest, excavations must be temporarily supported to maintain excavation stability and to limit potential adverse impacts on neighbouring structures. Structural support should be provided by suitable shoring (e.g. soldier pile, contiguous bored piles etc.) along affected excavation boundaries to maintain required stability.

Excavation may encounter groundwater seepage resulting from surface water infiltration. We expect sump pump methods to be adequate in managing expected inflow rates.

4.3 Retaining Structures

Retaining wall design should consider additional surcharge from live loads, construction equipment, new and existing infrastructure. Preliminary earth pressure coefficients for retaining wall design are provided in Section 4.4.

4.4 Earth Pressure Coefficients

Shoring or retaining wall design may adopt active, at rest and passive earth pressure coefficients of:

- 0.33, 0.50, 3.00 for uncontrolled fill.
- 0.39, 0.56, 2.56 for residual sandy clays.
- 0.26, 0.41, 3.86 for residual clayey sands.
- 0.30, 0.50, 3.00 for very low strength sandstone.

4.5 Foundations

<u>Swimming Pool</u>

Foundations for the new swimming pool are recommended to be supported by shallow footings such as slab on ground, founded on highly weathered, very low strength sandstone with an allowable bearing capacity of 400 kPa.

<u>Balcony Extension and Retaining Structures</u>

Due to presence of shallow bedrock, suitable foundations such as pad / strip footings should be founded in highly weathered, very low strength sandstone with an allowable bearing capacity of 400 kPa.

If higher end bearing capacity is required, we recommend bored piers socketed at least 0.5 m into highly weathered, very low to low strength sandstone with an allowable end bearing capacity of 800 kPa. The founding depth of piers should comply with AS2159 (2009) – Piling Design and Installation.

All foundations should be inspected by an experienced geotechnical engineer during construction stage to confirm encountered conditions satisfy design assumptions.

All footings should be founded on consistent material to limit potential of long-term differential settlement.

4.6 Drainage Requirements

Appropriate surface and sub-surface drainage should be provided to divert overland flows away from and limit ponding of water near footings and foundations.

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

4.7 Site Classification

The site is classified as a class 'P' site in accordance with AS 2870 (2011) due to presence of uncontrolled fill up to at least 0.6 mbgl. Reclassification to class 'A' may be possible subject to all foundation founding on sandstone bedrock.

5 Proposed Additional Works

5.1 Work Prior to Construction Certificate

We recommend the following additional geotechnical assessments are carried out to develop the final design and prior to construction:

1. If higher bearing capacities are required, additional investigation comprising deeper boreholes to bedrock will be required. This would include additional boreholes within the vicinity of the new swimming pool.

2. Review of the final design by a senior geotechnical engineer to confirm adequate consideration of the geotechnical risks and adoption of the recommendations provided in this report.

5.2 Construction Monitoring and Inspection

We recommend the following as summarised in Table 4 is inspected and monitored during construction of the project.

Table 4: Recommended inspection / monitoring requirements during site works.

Scope of Works	Frequency/Duration	Who to Complete
Inspect batters and excavation support (shoring and retaining wall) installations and batters and monitor associated performance to assess need for additional support requirements.	At 1 m depth intervals or as required [2]	Builder / MA [1]
Monitor groundwater seepage from excavation faces, if encountered, to assess suitability of exposed materials and need for additional drainage requirements.	When encountered	Builder / MA [1]
Inspect exposed material at foundation level to verify suitability as foundation.	Prior to reinforcement set-up and concrete placement, or fill placement	MA [1]
Monitor sedimentation downslope of excavated areas.	During and after rainfall events	Builder
Monitor sediment and erosion control structures to assess adequacy and for removal of built-up spoil.	After rainfall events	Builder

Notes:

1. MA = Martens and Associates engineer.
2. MA inspection frequency to be determined based on initial inspection findings in line with construction program.

6 References

Lone Pine Landscapes (2022), *Landscape Site Plan*, Drawing no. LPL_1001, LPL_1101, LPL_1102, LPL_1103, Landscape Sections and Elevations, Drawing no. LPL_3001 to LPL3003, Revision D, dated 15.02.2022 (LPL, 2022).

CMS Surveyors Pty Ltd (2022) *Survey Plan*, Sheet 1 of I, Dated 21.04.2022 (CMS, 2022).

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

Land and Property Information (2020) SIX Maps Viewer, (www.maps.six.nsw.gov.au).

NSW Department of Environment & Heritage (2021) *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.

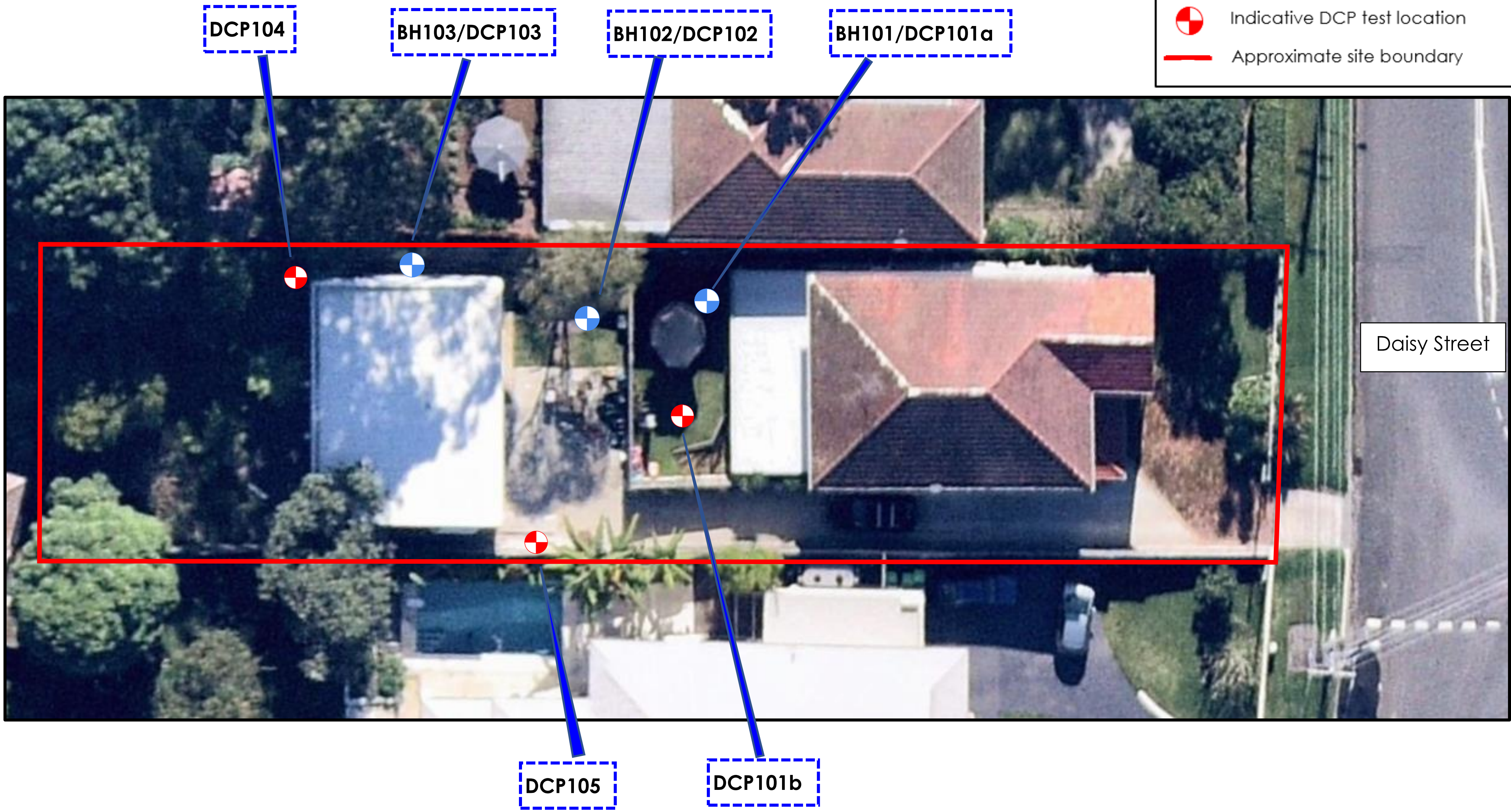
Standards Australia Limited (2017) AS 1726:2017, *Geotechnical site investigations*, SAI Global Limited.

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

Standards Australia Limited (2009) AS 2159:2009, Piling – Design and installation, SAI Global Limited.

Standards Australia Limited (2009) AS 3600:2009, *Concrete Structures*, SAI Global Limited.

Attachment A – Geotechnical Testing Plan



Key:

- Indicative borehole test location
- Indicative DCP test location
- Approximate site boundary

Martens & Associates Pty Ltd ABN 85 070 240 890		**Environment | Water | Wastewater | Geotechnical | Civil | Management**	
Drawn:	MZ	**GEOTECHNICAL TESTING PLAN** **10 Daisy Street, North Balgowlah, NSW** **(Nearmaps)**	**FIGURE 1**
Approved:	KB		
Date:	09.06.2022		Job No: P2209086JR01V01
Scale:	NA		

Attachment B – Borehole Logs

CLIENT	Joanna Ward	COMMENCED	26/07/2022	COMPLETED	26/07/2022	REF BH101
PROJECT	Geotechnical Assessment	LOGGED	MZ	CHECKED	KB	Sheet 1 OF 1
SITE	10 Daisy Street, North Balgowlah, NSW	GEOLOGY	Hawkesbury Sandstone	VEGETATION	Grass	PROJECT NO. P2209086
EQUIPMENT	Hand Auger	LONGITUDE	151.25444	RL SURFACE	45.5 m	DATUM AHD
EXCAVATION DIMENSIONS	Ø100 mm x 0.20 m depth	LATITUDE	-33.78873	ASPECT	East	SLOPE <5-10%

Drilling					Sampling		Field Material Description					
METHOD	PENETRATION RESISTANCE	WATER	DEPTH (metres)	*DEPTH* RL	SAMPLE OR FIELD TEST	RECOVERED	GRAPHIC LOG	USCS / ASCS CLASSIFICATION	SOIL/ROCK MATERIAL DESCRIPTION	MOISTURE CONDITION	CONSISTENCY DENSITY	STRUCTURE AND ADDITIONAL OBSERVATIONS
HA	L	Not Observed		45.50				SP	TOPSOIL: SAND; fine grained; dark brown to grey; with rootlets.	W	L	TOPSOIL
			0.2	*0.19* 45.31					Becomes light brown.			
									Hole Terminated at 0.20 m			0.20: Hand auger refusal on inferred very low to low strength sandstone.
			0.4									
			0.6									
			0.8									
			1.0									
			1.2									
			1.4									

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

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Engineering Log - BOREHOLE

MARTENS 2.00 LIB.GLB Log MARTENS BOREHOLE P2209086BH101-103.GPJ <<DrawingFile>> 04/08/2022 13:25 10.02.00.04 Datgel Lab and In Situ Tool - DGD | Lib: Martens 2.00 2016-11-13 Prj: Martens 2.00 2016-11-13

CLIENT	Joanna Ward	COMMENCED	26/07/2022	COMPLETED	26/07/2022	REF BH102	
PROJECT	Geotechnical Assessment	LOGGED	MZ	CHECKED	KB	Sheet 1 OF 1	
SITE	10 Daisy Street, North Balgowlah, NSW	GEOLOGY	Hawkesbury Sandstone	VEGETATION	Grass	PROJECT NO. P2209086	
EQUIPMENT	Hand Auger	LONGITUDE	151.25436	RL SURFACE	47.16 m	DATUM	AHD
EXCAVATION DIMENSIONS	Ø100 mm x 1.25 m depth	LATITUDE	-33.78872	ASPECT	East	SLOPE	<5-10%

Drilling					Sampling		Field Material Description					
METHOD	PENETRATION RESISTANCE	WATER	DEPTH (metres)	DEPTH RL	SAMPLE OR FIELD TEST	RECOVERED	GRAPHIC LOG	USCS / ASCS CLASSIFICATION	SOIL/ROCK MATERIAL DESCRIPTION	MOISTURE CONDITION	CONSISTENCY DENSITY	STRUCTURE AND ADDITIONAL OBSERVATIONS
HA	L-M		0.0	47.16	0.2-0.4/S/1 D 0.20-0.40 m			SM	FILL: Silty SAND; fine grained; dark grey, white, light brown, yellow brown; trace clay, rootlets and gravels.	W		FILL
			0.4	0.40 46.76	0.5-0.7/S/1 D 0.50-0.70 m			CI	Sandy CLAY; medium plasticity; pale brown to grey.	M (>PL)	St	RESIDUAL SOIL POSSIBLY FILL
		Inflow.	0.7	0.70 46.46	1.0-1.2/S/1 D 1.00-1.20 m			SC	Clayey SAND; fine grained; pale grey.	M - W	MD - D	RESIDUAL SOIL
			1.15	1.15 46.01					Becomes white.	W		
			1.25	1.25					Hole Terminated at 1.25 m			1.25: Hand auger refusal on inferred very low strength sandstone.

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

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Engineering Log - BOREHOLE

CLIENT	Joanna Ward	COMMENCED	26/07/2022	COMPLETED	26/07/2022	**REF BH103**	
PROJECT	Geotechnical Assessment	LOGGED	MZ	CHECKED	KB	Sheet 1 OF 1	
SITE	10 Daisy Street, North Balgowlah, NSW	GEOLOGY	Hawkesbury Sandstone	VEGETATION	Nil	PROJECT NO. P2209086	
EQUIPMENT	Hand Auger	LONGITUDE	151.25425	RL SURFACE	47.5 m	DATUM	AHD
EXCAVATION DIMENSIONS	Ø100 mm x 0.60 m depth	LATITUDE	-33.7887	ASPECT	East	SLOPE	<5-10%

Drilling					Sampling		Field Material Description					
METHOD	PENETRATION RESISTANCE	WATER	DEPTH (metres)	DEPTH RL	SAMPLE OR FIELD TEST	RECOVERED	GRAPHIC LOG	USCS / ASCS CLASSIFICATION	SOIL/ROCK MATERIAL DESCRIPTION	MOISTURE CONDITION	CONSISTENCY DENSITY	STRUCTURE AND ADDITIONAL OBSERVATIONS
HA	H			47.50					Mixed GRAVELS.			FILL
HA	M-H		0.2	0.10 47.40				SM	FILL: Silty SAND; fine grained; dark grey, white, light brown, yellow brown; trace clay and gravels.	M - W		
HA	M-H		0.4	0.40 47.10					With mixed gravels, brick fragments; trace mesh.			
HA	M-H	Inflow								W		
			0.6	0.60					Hole Terminated at 0.60 m			0.60: Hand auger refusal on inferred very low to low strength sandstone.
			0.8									
			1.0									
			1.2									
			1.4									

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

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Engineering Log - BOREHOLE

Attachment C – DCP ‘N’ Counts

Dynamic Cone Penetrometer Test Log Summary

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Site	10 Daisy Street, North Balgowlah, NSW	**DCP Group Reference**	P2209086JS01V01
Client	Joanna Ward	**Log Date**	26/07/2022
Logged by	MZ		
Checked by	KB		
Comments	DCPs commenced at 50 mm bgl.		

TEST DATA

Depth Interval (m)	DCP101a	DCP101b	DCP102	DCP103	DCP104	DCP105		
0.15	2	4	3	3	1	9		
0.30	Terminated due to double bounce @ 0.2 mbgl.	Terminated due to double bounce @ 0.25 mbgl.	6	5	Terminated due to double bounce on Sandstone bedrock @ 0 mbgl	5		
0.45			5	2		3		
0.60			6	Terminated due to double bounce @ 0.6 mbgl.		2		
0.75			6			3		
0.90			6			4		
1.05			9 / 720 mm			Terminated due to double bounce @ 0.97 mbgl.		
1.20			Terminated due to double bounce @ 1.08 mbgl.					
1.35								
1.50								
1.65								
1.80								
1.95								
2.10								
2.25								
2.40								
2.55								
2.70								
2.85								
3.00								
3.15								
3.30								
3.45								
3.60								
3.75								
3.90								
4.05								
4.20								
4.35								
4.50								
4.65								
4.80								

Attachment D – General Geotechnical Recommendations

Geotechnical Recommendations

Important Recommendations About Your Site (1 of 2)

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.

Attachment E – Notes About This Report

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)

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.

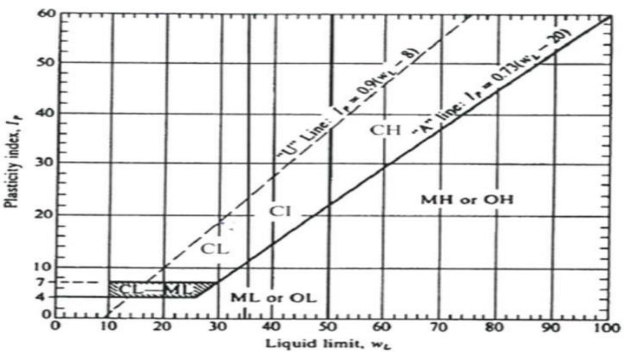
Particle Size

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

Symbols for Soils and Other

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)					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	(A 0.075 mm particle is about the smallest particle visible to the naked eye)	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 ≤35%; Medium Plasticity Clay – Liquid limit W_L >35%, ≤50%; High Plasticity Clay - Liquid limit W_L > 50%.
3. Low Plasticity Silt – Liquid Limit W_L ≤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.

Soil Data

martens consulting engineers

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.

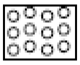
Symbol	Field Texture Grade	Behaviour of moist bolus	Ribbon length	Clay content (%)
S	Sand	Coherence nil to very slight; cannot be moulded; single grains adhere to fingers	0 mm	< 5
LS	Loamy sand	Slight coherence; discolours fingers with dark organic stain	6.35 mm	5
CLS	Clayey sand	Slight coherence; sticky 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

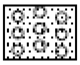
Rock Data

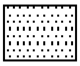
Symbols for Rock

SEDIMENTARY ROCK

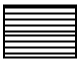
 BRECCIA

 CONGLOMERATE

 CONGLOMERATIC SANDSTONE

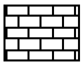
 SANDSTONE/QUARTZITE

 SILTSTONE

 MUDSTONE/CLAYSTONE

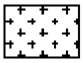
 SHALE

 COAL

 LIMESTONE

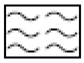
 LITHIC TUFF

IGNEOUS ROCK

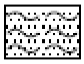
 GRANITE

 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

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

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.

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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

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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	◁ Partial water loss
▷ Water inflow	◀ 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		