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By Hand ☐
Contact: Ralph Erni
Our Ref: P1907220JC01V01
Pages: 1+ 2 Attachments
cc. -

22 May 2019

COMMPLAN PTY LTD
Attn: Emma Lachlan

By email

Dear Emma,

RE: PROPOSED NEW TELECOMMUNICATIONS FACILITY: OPTUS SITE S3176 – A (BLUE COW), SKI TUBE TERMINAL, BLUE COW, NSW.

Martens and Associates (MA) has reviewed design drawings referenced S3176-P1 and P2 Revision 01, prepared by Optus, for proposed modifications to an existing Telstra telecommunication facility and construction of a new raised equipment shelter at the above site.

MA is satisfied that the design drawings meet the recommendations presented in MA's Geotechnical Assessment report reference P1605378JR01V01 dated November 2016. We recommend that the development is design and constructed in accordance with this report.

We have attached the Form 4 – Minimal Impact Certification in accordance with the NSW Planning & Environment Geotechnical Policy for Kosciuszko Alpine Resorts.

If you require any further information, please do not hesitate to contact the undersigned.

For and on behalf of

MARTENS & ASSOCIATES PTY LTD

RALPH ERNI
B.Sc. Eng (Civil) M.Eng. (Geo) CPEng NER
Senior Geotechnical Engineer

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MARTENS & ASSOCIATES P/L
ABN 85 070 240 890 ACN 070 240 890

NSW GOVERNMENT | Planning & Environment

Geotechnical Policy
Kosciuszko Alpine Resorts

Form 4 – Minimal Impact Certification

DA Number: ____________________

This form may be used where minor construction works which present minimal or no geotechnical impact on the site or related land are proposed to be erected within the "G" line area of the geotechnical maps.

A geotechnical engineer or engineering geologist must inspect the site and/or review the proposed development documentation to determine if the proposed development requires a geotechnical report to be prepared to accompany the development application. Where the geotechnical engineer determines that such a report is not required then they must complete this form and attach design recommendations where required. A copy of Form 4 with design recommendation, if required, must be submitted with the development application.

Please contact the Alpine Resorts Team in Jindabyne for further information - phone 02 6456 1733.

To complete this form, please place a cross in the appropriate boxes ☐ and complete all sections.

1. Declaration made by geotechnical engineer or engineering geologist in relation to a nil or minimal geotechnical impact assessment and site classification

I,

Mr ☒ Ms ☐ Mrs ☐ Dr ☐ Other ☐

First Name: RALPH

Family Name: ERNI

OF

Company/organisation: MARTENS & ASSOCIATES

certify that I am a geotechnical engineer /engineering geologist as defined by the "Policy" and I have ~~inspected the site~~ reviewed and approved a geotechnical assessment for site development in 2016 and reviewed the proposed development known as,

OPTUS SITE S3176 BLUE COW

As a result of my site inspection and review of the following documentation

(List of documentation reviewed)

OPTUS DWGS S3176-P1 & P2 Rev 01, 06.04.2017

MARTENS GEOTECHNICAL INVESTIGATION REPORT

referenced P1605378 JR01 V01 dated Nov 2016

I have determined that;

☐ the current load-bearing capacity of the existing building will not be exceeded or adversely impacted by the proposed development, and

☒ the proposed works are of such a minor nature that the requirement for geotechnical advice in the form of a geotechnical report, prepared in accordance with the "Policy", is considered unnecessary for the adequate and safe design of the structural elements to be incorporated into the new works, and

☒ in accordance with AS 2870.1 Residential Slabs and Footings, the site is to be classified as a type

(insert classification type)

A, if founding on rock, otherwise S.

☒ I have attached design recommendations to be incorporated in the structural design in accordance with this site classification. Refer MA report P1605378JR01V01 dated Nov 2016

I am aware that this declaration shall be used by the Department as an essential component in granting development consent for a structure to be erected within the "G" line area (as identified on the geotechnical maps) of Kosciuszko Alpine Resorts without requiring the submission of a geotechnical report in support of the development application.

2. Signatures

Signature

Chartered professional status

CPENG NER

Name

RALPH ERNI

Date

21/5/2019

3. Contact details

Alpine Resorts Team
Shop 5A, 19 Snowy River Avenue
P O Box 36, JINDABYNE NSW 2627
Telephone: 02 6456 1733
Facsimile: 02 6456 1736
Email: alpineresorts@planning.nsw.gov.au

NSW GOVERNMENT | Planning & Environment

Geotechnical Policy

Kosciuszko Alpine Resorts

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DA Number: ____________________

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First Name: RALPH

Family Name: ERNI

OF

Company/organisation: MARTENS & ASSOCIATES

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OPTUS SITE S3176 BLUE COW

As a result of my ~~site inspection and~~ review of the following documentation

(List of documentation reviewed)

OPTUS DWGS S3176-P1 & P2 Rev 01, 06.04.2017

MARTENS GEOTECHNICAL INVESTIGATION REPORT referenced P1605378JR01V01 dated Nov 2016

I have determined that;

- ☐ the current load-bearing capacity of the existing building will not be exceeded or adversely impacted by the proposed development, and
- ☒ the proposed works are of such a minor nature that the requirement for geotechnical advice in the form of a geotechnical report, prepared in accordance with the "Policy", is considered unnecessary for the adequate and safe design of the structural elements to be incorporated into the new works, and
- ☒ in accordance with AS 2870.1 Residential Slabs and Footings, the site is to be classified as a type

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2. Signatures

Signature

Chartered professional status: CPENG NER

Name: RALPH ERNI

Date: 21/5/2019

3. Contact details

Alpine Resorts Team
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Kordia Solutions Pty Ltd

Geotechnical Investigation: TELSTRA SITE 41627 KOSCIUSZKO NTNL PARK BLUE COW - Perisher Blue Cow Link Rd, Kosciuszko National Park, NSW

P1605378JR01V01
November 2016

ENVIRONMENTAL

WATER

WASTEWATER

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Document and Distribution Status

Author(s)	Reviewer(s)	Project Manager	Signature
Sam Ziu	Ralph Erni	Gray Taylor	

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				File Copy	Kordia Solutions Pty Ltd		
1	Geotechnical Investigation	Final	16.11.2016	1E,1P,1H	1P		

Distribution types: F = Fax, H = Hard copy, P = PDF document, E = Other electronic format. Digits indicate number of document copies.

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

Contents

1 Introduction

1.1 Overview

This report documents the findings of a geotechnical investigation carried out to support the design of a proposed telecommunications installation (20m high Custom Lattice Tower) to be located at Perisher Blue Cow Link Rd, Kosciuszko National Park, NSW (TELSTRA SITE 41627 KOSCIUSZKO NTNL PARK BLUE COW).

This report has been prepared in general accordance with AS1726 (1993), the requirements of the Client and the agreed scope of work. It provides descriptions of sub-surface conditions encountered during field investigations, with corresponding geotechnical design parameters and site classification. In addition, this report provides in-situ soil resistivity and point load test results.

1.2 Field Investigations

Field investigations conducted on 2 November 2016 included:

- General site walkover to assess existing site conditions including local topography, geology, exposed soil conditions, drainage and vegetation.
- Review of DBYD survey plans.
- Drilling of a borehole (BH101) at an accessible location as close as possible to the proposed development to characterise subsurface materials. The borehole was drilled with a 4WD truck-mounted hydraulic rig using solid flight augers fitted with a V-shaped bit (V-bit) to 0.3 metres below ground level (mBGL), and a Tungsten Carbide bit (TC-bit) to 2.50 mBGL, followed by NMLC rock coring to a depth of 6.00 mBGL.
- One Dynamic Cone Penetrometer (DCP) test (DCP101) to 0.30 mBGL to assist soil characterisation and estimation of soil strength parameters in accordance with AS 1289.6.3.2 (1997).
- Soil resistivity testing using an AEMC 4620 Ground Resistance Tester and adopting the Wenner 4 pin method in accordance with Standards Australia HB 160 (2006).
- Logging of the borehole and collection of soil and rock samples for future reference.

Approximate borehole, DCP test and resistivity test transect locations are shown in Figure 1, Attachment A.

2 Geotechnical Assessment

2.1 Site Conditions

Table 1 summarises general site conditions considered relevant to the investigation.

Table 1: General site conditions.

Element	Description/Detail
Topography	The site is located within mountainous terrain in the Australian Alps in the Kosciuszko National Park. It is located in highly undulating terrain. It is heavily vegetated and contains several large granite outcrops. Slopes are generally between 5% and 30%.
Expected Geology	Lower Devonian (Dlg) mainly concordant gneissic to massive magmatic intrusives including Kosciusko Granites (Adamson C.L., Browne W.R., Carne J.E., Den Tex E. and Vallance T. G., 1966, *Tallangatta 1:250 000 Geological Sheet SJ 55-3*, Geological Survey of New South Wales, N.S.W. Department of Mines)
Site Aspect	East
Elevation	1904 mAHD (based on drawing No. N25639 Sheet No. S3-1 provided by Kordia Solutions Pty Ltd)
Typical Slope	Generally less than 2 % in drilling area
Existing Vegetation	Grass/shrubs
Site Drainage	Via overland flow towards east
Surrounding Conditions	Surrounded by bushland on all sides. Perisher Blue Cow link Road is located to north west of the site.

2.2 Sub-surface Conditions

Table 2 summarises encountered sub-surface materials and conditions, inferred from borehole and field test results, to investigation termination depth. Encountered conditions are described in more detail on the borehole log in Attachment B, and associated explanatory notes in Attachment F. For DCP test results refer to DCP "N" counts in Attachment C.

Table 2: Generalised description of inferred sub-surface profile.

Layer Description [1]	Depth [2] (mBGL) BH101
TOPSOIL: Silty SAND (fine to medium grained with trace of granite, gravels and organics, medium dense, moist)	0.0 – 0.15
RESIDUAL: Silty SAND (fine to medium grained with trace of granite and gravels, very dense, moist)	0.15 – 0.30 [3]
WEATHERED ROCK: GRANITE (inferred very low to low strength, distinctly weathered, moist)	0.3 – 2.5 [4, 5]
WEATHERED ROCK: GRANITE (medium to high strength, moderately to slightly weathered)	2.5 – 6.00 [6]

Notes:

1. Refer to the borehole log in Attachment B for more detailed material descriptions at test location.
2. Indicative depth range below ground level, to end of borehole, which may vary across site depending on site and local geological conditions.
3. V-bit refusal on inferred very low to low strength granite.
4. TC-bit refusal in inferred medium strength granite. Rock coring commenced.
5. Water inflow encountered at 1.5 mBGL.
6. Rock core between approximately 5.85 mBGL and 6.00 mBGL could not be retrieved and remained in hole.

2.3 Groundwater

Soils were encountered in a generally moist condition up to 1.5 mBGL and then wet to investigation termination depth. Groundwater inflow was encountered at 1.5 mBGL. The introduction of drilling fluids during rock coring at 2.5 mBGL prevented observation of groundwater inflow below. We have adapted a groundwater level of 1.5 mBGL for the purpose of this report. Should further information on permanent site groundwater conditions be required, additional assessment would need to be carried out (i.e. installation of groundwater monitoring bore).

2.4 Rock Coring and Laboratory Test Results

Three rock core samples were collected from BH101 and submitted to Resource Laboratories, a National Association of Testing Authorities (NATA) accredited laboratory. The samples were subject to point load strength index testing, undertaken for the purpose of characterising encountered rock. Laboratory test results are summarised in Table 3 and the laboratory test certificate is provided in Attachment E. A rock core photo is provided in Attachment D.

Table 3: Point load strength index testing results.

Borehole	Sample Depth (mBGL)	Point Load Strength Index $I_{s(50)}$ (MPa)		UCS [1] (MPa)	Rock Strength [2]
		Diametral	Axial		
101	2.80	0.94	0.93	18.6	Medium-High
101	3.90	0.92	0.90	18.0	Medium-High
101	5.40	1.10	0.70	14.0	Medium-High

Notes:

1. Unconfined Compressive Strength of intact material, assuming UCS = 20* $I_{s(50)}$, considering axial load direction.
2. Strength classification based on AS1726 (1993).

Test results and observations during rock coring indicate that the bedrock at BH101 consists of very low to low strength granite between 0.30 mBGL and 2.50 mBGL, and medium to high strength granite between approximately 2.50 and 6.00 mBGL. It should be considered that testing was carried out on relatively intact core samples. Engineering properties of the rock mass will be impacted by the presence of defects in the rock profile, including weathered zones.

2.5 Soil Resistivity

The results of in-situ soil resistivity testing are summarised in Table 4.

Table 4: Soil resistivity test data.

Test No.	E – W Transect [1]			N – S Transect [1]		
	Rod Spacing (A) (m)	Rod Depth (B) (m)	Measured Ohms (R) (Ω)	Rod Spacing (A) (m)	Rod Depth (B) (m)	Measured Ohms (R) (Ω)
1	8	0.20	High Res [2]	8	0.20	High Res [2]
2	4	0.20	High Res [2]	4	0.20	High Res [2]
3	2	0.10	High Res [2]	2	0.10	High Res [2]
4	1	0.05	High Res [2]	1	0.05	High Res [2]

Notes:

1. Refer to site plan in Attachment A for indicative transect alignments.
2. Reading exceeds instrument measurement limits.

3 Recommendations

3.1 Geotechnical

3.1.1 Proposed Footing Systems and Foundation Levels

We recommend adopting the following for design of footings for proposed structures.

New lightly loaded high-level structures:

- Residual soil with traces of organics (topsoil) is considered unsuitable as foundation material. These materials should be removed, if necessary, and replaced with 'engineered' fill (under 'engineered' condition). Fill materials, earthworks and compliance testing should be in accordance with AS3798 (2007).
- Shallow footings including slab-on-ground with thickened edge beams may be adopted if founded on residual soil without organics and/or on granite. To limit differential movements, it is recommended that footings are founded on material with similar end bearing capacity.

New monopole:

- Pad footing may be adopted founding on at least low strength granite. Pad size may be reduced by inclusion of tie-down anchors. If adapted, further advice should be sought from a geotechnical engineer.
- Alternatively, deepened footing, such as a bored cast in-situ concrete pile may be considered. However, consideration should be given to the likely presence of very high strength granite below investigation termination depth.

Shallow footing and pile designs should consider recommended preliminary design parameters in Table 6, which are to be confirmed by a geotechnical engineer during construction stage.

Excavations for footings should be viewed by a geotechnical engineer prior to footing construction with minimal delay following excavation completion. The geotechnical engineer is to confirm encountered sub-surface conditions satisfy design assumptions and that the bases of excavations are free from loose or softened material and water prior to footing construction. Water that has ponded in the bases of excavations and any resultant softened materials are to be removed prior to footing construction. If a delay in shallow footing construction is anticipated, we recommend that a concrete blinding layer of at least 50 mm thickness is placed to protect the foundation material.

3.1.2 Preliminary Material Properties and Design Parameters

Preliminary material properties inferred from observations during borehole drilling, such as auger penetration resistance, and DCP test results are summarised in Table 5. Table 6 summarises geotechnical design parameters for encountered sub-surface conditions recommended for design of new shallow footings for the equipment shelter and piles or pad footing for the new monopole.

Design parameters in Table 6 assume the base of excavation is free of loose or soft soils and water prior to placement of concrete and approved following inspection by an experienced geotechnical engineer.

Table 5: Preliminary material properties.

Layer	$\gamma_{in\text{-}situ}$ [1] (kN/m^3)	UCS [2] (MPa)	Ø' [3] (deg)	E' [4] (MPa)	K_s [5] (MPa/m)
RESIDUAL [7]: Silty SAND (fine to medium grained very dense, moist)	20	NA [8]	40	40	40
WEATHERED ROCK: GRANITE (inferred very low to low strength, distinctly weathered, moist)	23	3	40	100	80
WEATHERED ROCK: GRANITE (medium to high strength, moderately to slightly weathered) [6]	24	15	42	500	400

Notes:

1. Material in-situ unit weight, based on visual assessment (±10 %).
2. Unconfined compressive strength of intact material (assumed average for unit).
3. Effective internal friction angle (±2 °) assuming drained conditions, may be dependent on rock defect conditions.
4. Effective elastic modulus (±10 %), that should be adopted to calculate lateral deflection of pile in soil / rock under serviceability loading.
5. Modulus of subgrade reaction (vertical). For horizontal modulus, 1/3 vertical Ks may be adopted.
6. High strength rock may be present below investigation termination depth.
7. Assuming topsoil is removed from the development footprint.
8. Not applicable.

Table 6: Recommended geotechnical design parameters.

Layer	Shallow Footings	Piles [1]			K_a [6]	K_p [6]
	ABC [2, 5]	ABC [2, 5]	ALBC [3,5]	ASF [4, 5]		
RESIDUAL: Silty SAND (fine to medium grained, very dense)	100	NA [9]	NA [9]	NA [9]	0.30	3.33
WEATHERED ROCK: GRANITE (inferred very low to low strength, distinctly weathered)	500	1000	700	150	NA [9]	NA [9]
WEATHERED ROCK: GRANITE (medium to high strength, moderately to slightly weathered)	NA [9]	3000	2000	300	NA [9]	NA [9]

Notes:

1. Assuming bored cast in-situ pile.
2. Allowable end bearing capacity (kPa) for footings embedded at least 0.3 m for lightly loaded footings, or alternatively at least 0.5 m for larger pads (e.g. monopole footing), and piles embedded at least 0.5 m or 1 pile diameter, whichever is greater, into design material type subject to confirmation on site by a geotechnical engineer of inferred foundation conditions.
3. Allowable lateral bearing capacity (kPa).
4. Allowable skin friction (kPa) below 1 m depth for bored pile in compression, assuming intimate contact between pile and foundation material. For up lift resistance, we recommend reducing ASF by 50% and checking against 'piston' and 'cone' pull-out mechanisms in accordance with AS2159 (2009).
5. ABC and ASF are given with estimated factors of safety of 3 and 2 respectively, generally adopted in geotechnical practice to limit settlement to an acceptable level for conventional building structures and to 25 mm for a large single pad footing.
6. K_a = Coefficient of active earth pressure; K_p = Coefficient of passive earth pressure.
7. Assuming lightly loaded high level structures supported by square footing with $D_f/B < 0.5$, $D_f > 0.3$ m.
8. Assuming large pad footing with $D_f/B < 0.5$, $D_f > 0.8$ m and $B < 4$ m.
9. Not applicable or side adhesion not recommended either due to shallow depth or potential internal settlement of materials.

3.1.3 Site Classification

The site is classified as an "A" site, in accordance with AS 2870 (2011), for design of lightly loaded shallow footings founding on sand or granite.

This site classification is subject to the recommendation presented in this report and the following conditions:

- Footings extend through all topsoil, uncontrolled fill or root affected soils.
- Requirement for only minor changes to current site levels.

- Provision of adequate drainage of surface and sub-surface water to limit soil moisture variations impacting on foundation conditions.

- Footings are unlikely to be impacted by the presence of environments that could lead to exceptional foundation material movements, such as existing or future trees or surface water accumulation.

3.2 Construction Considerations

Excavations exceeding 0.75 m should be battered back at slopes of no greater than 1 V (vertical): 2 H (horizontal) for temporary slopes, or retained. Unsupported excavations deeper than 1.0 m should be assessed on site by a geotechnical engineer for slope instability risk. Use of heavy machinery should be avoided, where possible, within 2 m of the crest of any soil excavation to prevent excessive vibrations and undue settlement within exposed soils.

All excavation work should be completed with reference to the Code of Practice 'Excavation Work' (July 2015), by Safe Work Australia, and excavation requirements of Council's Development Control Plan.

We expect that sump and pump methods will be appropriate for collection and removal of surface water and potential ephemeral perched groundwater inflow, if encountered during construction of shallow footings. Pile excavations may encounter groundwater inflow, however, we expect inflow rates will be manageable by pumping or using a trammie system for placement of concrete.

All surface runoff should be diverted away from excavation areas during construction works and foundations to prevent foundation material strength reduction as a result of soil saturation and limit soil erosion.

4 Limitations

The recommendations presented in this report are based on limited preliminary investigations and include specific issues to be addressed during the design and construction phases of the project. In the event that any of the recommendations presented in this report are not implemented, the general recommendations may become inapplicable and Martens & Associates accept no responsibility whatsoever for the performance of the works undertaken where recommendations are not implemented in full and properly tested, inspected and documented.

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

Relative ground surface level at the borehole location is based on data from drawing number N25639 Sheet No. S3-1 provided by Kordia Solutions Pty Ltd.

5 References

AEMC Instruments Workbook Edition 7.0 (2000) *Understanding Ground Resistance Testing*, 950.WKBK-GROUND 08/00.

AEMC Instruments (2012) *Digital Ground Resistance Testers 4620 and 4630, User Manual*, 99-MAN 100259v13, Section 5.5.

Adamson C.L., Browne W.R., Carne J.E., Den Tex E. and Vallance T. G., 1966, *Tallangatta 1:250 000 Geological Sheet SJ 55-3*, Geological Survey of New South Wales, N.S.W. Department of Mines.

Drawing No. N25639 Sheet No. S3-1 drawn by Ubris Pty Ltd

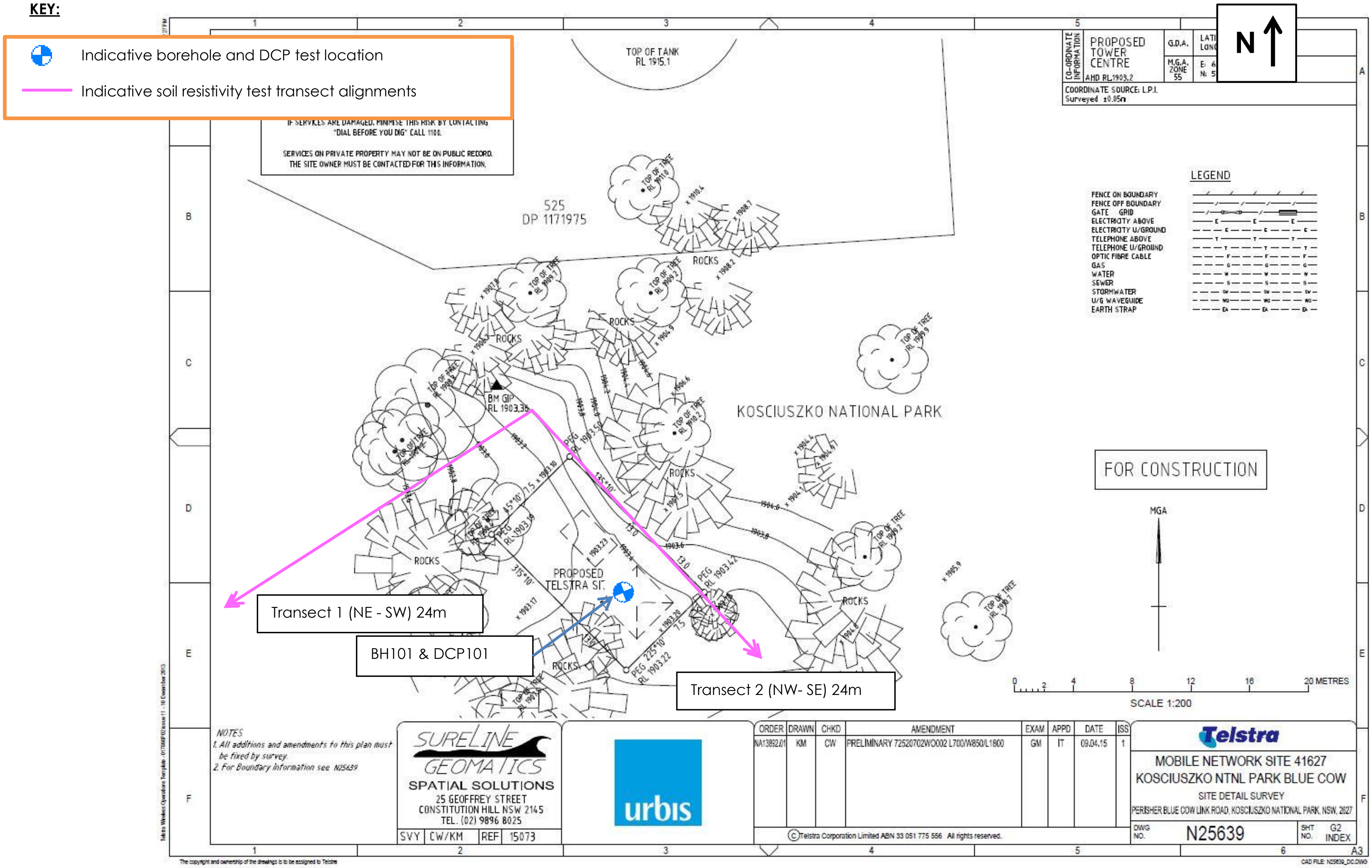
Standards Australia AS1289.6.3.2 (1997) *Determination of the Penetration Resistance of a Soil – 9 kg Dynamic Cone Penetrometer Test*.

Standards Australia AS1726 (1993) *Geotechnical Site Investigations*.

Standards Australia AS2870 (2011) *Residential Slabs and Footings*.

Standards Australia AS3600 (2009) *Concrete Structures*.

6 Attachment A – Site Layout and Geotechnical Testing Plan



Martens & Associates Pty Ltd ABN 85 070 240 890		**Environment | Water | Wastewater | Geotechnical | Civil | Management**	
Drawn:	SZ	**SITE LAYOUT AND GEOTECHNICAL TESTING PLAN** **TELSTRA SITE 41627 - KOSCIUSZKO NTNL PARK BLUE COW** Perisher Blue Cow Link Rd, Kosciuszko National Park, NSW (Source: Kordia Solutions Pty Ltd, Drawing No. N25639 Sheet No. G2)	Drawing: **FIGURE 1**
Approved:	RE		
Date:	16.11.2016		
Scale:	NA		File No: P1605378JR01V01

7 Attachment B – Test Borehole Log

CLIENT	Kordia Solutions Pty Ltd	COMMENCED	02/11/2016	COMPLETED	12/11/2016	REF BH101
PROJECT	Geotechnical Investigation	LOGGED	HD	CHECKED	RE	Sheet 1 OF 3
SITE	Telstra Site 41627 KOSCIUSZKO NTNL PARK, NSW	GEOLOGY	Kosciusko Granite	VEGETATION	Grass/shrubs	PROJECT NO. P1605378
EQUIPMENT	4WD truck-mounted hydraulic drill rig	EASTING		RL SURFACE	1904 m	DATUM AHD
EXCAVATION DIMENSIONS	Ø100 mm x 6.00 m depth	NORTHING		ASPECT	East	SLOPE <2%

Drilling					Sampling		Field Material Description					
METHOD	PENETRATION RESISTANCE	WATER	DEPTH (metres)	DEPTH RL	SAMPLE OR FIELD TEST	RECOVERED	GRAPHIC LOG	USCS SYMBOL	SOIL/ROCK MATERIAL DESCRIPTION	MOISTURE CONDITION	CONSISTENCY DENSITY	STRUCTURE AND ADDITIONAL OBSERVATIONS
AD/V	L			1904.00 0.15			▨	SM	Silty SAND, fine to medium grained, dark brown with some fine to medium grained subangular to angular granite gravels, trace of organics.	M	MD	TOPSOIL
				1903.85 0.30			× · · ×	SM	Silty SAND, fine to medium grained, dark brown with some fine to medium grained subangular to angular granite gravels, trace of organics.		VD	RESIDUAL SOIL
AD/T			0.5 1.0	1903.70 1.50	5378/101/0.30/S/1 D 0.30 m 5378/101/0.50/R/1 D 0.50 m		+ +		GRANITE, fine to medium grained (0.5-1.0mm), mottled dark grey, pink and clear (off white), inferred very low to low strength, distinctly weathered.			WEATHERED ROCK 0.30: V-bit refusal.
		▽ Water inflow	1.5 2.0	1902.50 2.50	5378/101/2.0/R/1 D 2.00 m		+ +		GRANITE, fine to medium grained (0.5-1.0mm), dark grey, some pink and clear (off white), inferred very low to low strength, distinctly weathered.	W		
			2.5 3.0 3.5 4.0 4.5						Continued as Cored Borehole			

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

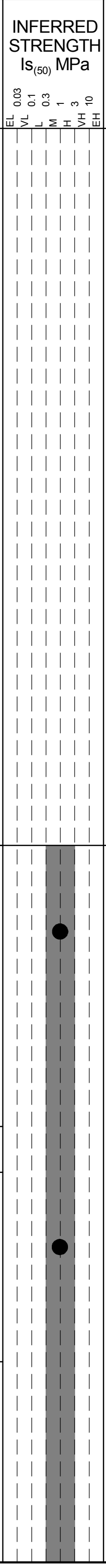
martens

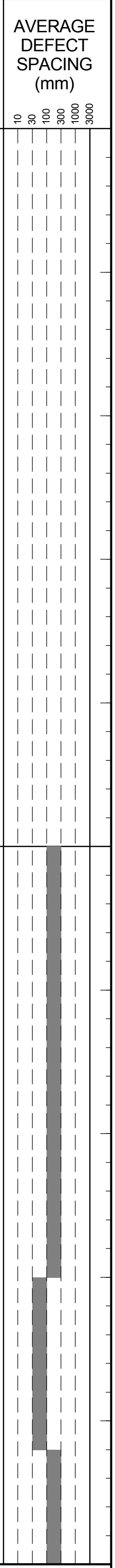
MARTENS & ASSOCIATES PTY LTD
Suite 201, 20 George St. Hornsby, NSW 2077 Australia
Phone: (02) 9476 9999 Fax: (02) 9476 8767
mail@martens.com.au WEB: http://www.martens.com.au

Engineering Log - BOREHOLE

CLIENT	Kordia Solutions Pty Ltd	COMMENCED	02/11/2016	COMPLETED	12/11/2016	**REF BH101**
PROJECT	Geotechnical Investigation	LOGGED	HD	CHECKED	RE	Sheet 2 OF 3
SITE	Telstra Site 41627 KOSCIUSZKO NTNL PARK, NSW	GEOLOGY	Kosciusko Granite	VEGETATION	Grass/shrubs	PROJECT NO. P1605378
EQUIPMENT	4WD truck-mounted hydraulic drill rig	EASTING		RL SURFACE	1904 m	DATUM AHD
EXCAVATION DIMENSIONS	Ø100 mm x 6.00 m depth	NORTHING		ASPECT	East	SLOPE <2%

Drilling						Field Material Description				Defect Information	
METHOD	WATER	TCR	RQD (SCR)	DEPTH (metres)	*DEPTH* RL	GRAPHIC LOG	ROCK / SOIL MATERIAL DESCRIPTION	WEATHERING	INFERRED STRENGTH Is(50) MPa	DEFECT DESCRIPTION & Additional Observations	AVERAGE DEFECT SPACING (mm)
				0.5							
				1.0							
				1.5							
				2.0							
				2.5	*2.50*		Continuation from non-cored borehole				
NMLC		100	77 (98)		1901.50	+ +	GRANITE, medium grained (2.0-5.0mm), grey white.	SW		2.56: Joint, 5°, VNR, UN, RF, Fine to medium sand. 2.70: DB 2.86: Joint, 10 - 20°, VNR, UN, RF, Fine sand.	
				3.0						3.00: HB 3.07: JT, 0 - 5°, VNR, UN, RF, Fine sand.	
				3.5				MW		3.51-3.85: CS, 0 - 5°, UN, RF, Coarse and fine to coarse sand. 3.60-3.72: JT, 60 - 70°, VNR, UN, RF, Coarse sand.	
								SW			
				4.0						4.00: JT, 5 - 10°, VNR, UN, RF, 40mm, fine to medium sand. 4.02-4.03: JT, 5 - 10°, CT, UN, RF, 1-2mm, fine to medium sand. 4.18-4.22: JT, 50°, UN, RF, 1-2mm, closed. Does not extend through entire diameter of core. 4.26-4.64: JT set, 10 - 20°, VNR, UN, RF, 40-70mm, fine to medium sand.	
								MW			
				4.5						4.66-4.76: JT, 70 - 80°, VNR, UN, RF, Fine to medium sand.	





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

Engineering Log - BOREHOLE

CLIENT	Kordia Solutions Pty Ltd	COMMENCED	02/11/2016	COMPLETED	12/11/2016	**REF BH101**	
PROJECT	Geotechnical Investigation	LOGGED	HD	CHECKED	RE	Sheet 3 OF 3	
SITE	Telstra Site 41627 KOSCIUSZKO NTNL PARK, NSW	GEOLOGY	Kosciusko Granite	VEGETATION	Grass/shrubs	PROJECT NO. P1605378	
EQUIPMENT	4WD truck-mounted hydraulic drill rig	EASTING		RL SURFACE	1904 m	DATUM	AHD
EXCAVATION DIMENSIONS	Ø100 mm x 6.00 m depth	NORTHING		ASPECT	East	SLOPE	<2%

Drilling						Field Material Description				Defect Information	
METHOD	WATER	TCR	RQD (SCR)	DEPTH (metres)	*DEPTH* RL	GRAPHIC LOG	ROCK / SOIL MATERIAL DESCRIPTION	WEATHERING	INFERRED STRENGTH $Is_{(50)}$ MPa EL 0.03 VL 0.1 L 0.3 M 1 H 3 VH 10 EH	DEFECT DESCRIPTION & Additional Observations	AVERAGE DEFECT SPACING (mm) 10 30 100 300 1000 3000
NMLC		100	77 (98)	5.5	*6.00* 1898.00	+ +	GRANITE, medium grained (2.0-5.0mm), grey white.	MW		5.00: HB 5.05-5.09: JT, 30 - 40°, VNR, UN, RF, Fine to medium sand. 5.30-5.31: JT, 0 - 5°, VNR, UN, RF, Fine clayey sand. 5.65-5.86: JT set 2, 75 - 85°, CT, UN, RF, Silt, 1-2mm. 5.86: JT, 0 - 10°, VNR, IR, RF, Fine to medium sand, 10-20mm. 5.86-6.00: Core could not be retrieved- remained in hole.	
				6.0 6.5 7.0 7.5 8.0 8.5 9.0 9.5			Hole Terminated at 6.00 m				

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

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Engineering Log - BOREHOLE

8 Attachment C – DCP "N" Counts

Dynamic Cone Penetrometer Test Log Summary

martens
consulting engineers since 1989

Suite 201, 20 George Street, Hornsby, NSW 2077 Ph: (02) 9476 9999 Fax: (02) 9476 8767, mail@martens.com.au, www.martens.com.a u

Site	TELSTRA SITE 41627 KOSCIUSZKO NTNL PARK BLUE COW Perisher Blue Cow Link Rd, Kosciuszko National Park, NSW	**DCP Group Reference**	P1605378JS01V01
Client	Kordia Solutions Pty Ltd	**Log Date**	02.11.2016
Logged by	HD		
Checked by	PS		
Comments			

TEST DATA

Depth Interval (m)	DCP101							Design
0.15	5							**5**
0.30	40							**40**
0.45	Terminated at 0.30 mBGL due to high 'N' counts							
0.60								
0.75								
0.90								
1.05								
1.20								
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								
4.95								
5.10								
5.25								
5.40								
5.55								
5.70								
5.85								
6.00								
6.15								
6.30								
6.45								
6.60								
6.75								
6.90								
7.05								
7.20								
7.35								
7.50								
7.65								
7.80								
7.95								
8.10								
8.25								
8.40								

9 Attachment D – Rock Core Photo

MARTENS & ASSOCIATES P/L

PROJECT: P1605378

BOREHOLE: BH101 DEPTH: 2.50m – 6.00 m DATE: 02.11.2016

Martens & Associates Pty Ltd ABN 85 070 240 890		**Environment | Water | Wastewater | Geotechnical | Civil | Management**	
Drawn:	SZ	**PHOTO OF ROCK CORE** **TELSTRA SITE 41627** **KOSCIUSZKO NTNL PARK BLUE COW** Perisher Blue Cow Link Rd, Kosciuszko National Park, NSW	Drawing: **FIGURE 1**
Approved:	RE		
Date:	16.11.2016		
Scale:	NA		File No: P160537818JR01V01

10 Attachment E – Lab Test Results

resource laboratories
AGGREGATE, ROCK, AND SOIL TESTING
ABN: 25 131 532 020
Sydney: 12/1 Boden Road Seven Hills NSW 2147 | PO Box 45 Pendle Hill NSW 2145
Ph: (02) 9674 7711 | **Fax:** (02) 9674 7755 | **Email:** info@resourcelab.com.au

Test Report

Customer: Martens & Associates Pty Ltd
Project: P1605378
Location: TELSTRA SITE 41627
KOSCIUSZKO NTNL PARK BLUE COW
Perisher Blue Cow Link Rd, Kosciuszko National Park, NSW

Job number: 16-0112
Report number: 1 rev.1
Page: 1 of 1

Point Load Strength Index

Sampling method: Samples tested as received

Test method(s): AS 4133.4.1 Clause 3.2, 3.3

	Results				
Laboratory sample no.	9979	9980	9981		
Customer sample no.	5378/101/2.80m/R/1	5378/101/3.90m/R/1	5378/101/5.40m/R/1		
Sample depth	2.80m	3.90m	5.40m		
Date sampled	12/11/2016	12/11/2016	12/11/2016		
Date tested	15/11/2016	15/11/2016	15/11/2016		
Lithological description	GRANODIORITE	GRANODIORITE	GRANODIORITE		
Diametral					
Moisture content condition	Dry	Dry	Dry		
Nature of weakness planes	n/a	Fracture	n/a		
Specimen size					
Length (mm)	165.0	159.0	174.0		
Diameter (mm)	51.1	51.3	51.2		
I_s (MPa)	**0.94**	**0.92**	**1.1**		
$I_{s(50)}$ (MPa)	**0.95**	**0.93**	**1.1**		
Failure mode	Through fabric of specimen	Along fracture	Through fabric of specimen		
Axial					
Moisture content condition	Dry	Dry	Dry		
Nature of weakness planes	n/a	n/a	n/a		
Specimen size					
Height (mm)	36.4	38.7	37.8		
Diameter (mm)	51.1	51.3	51.2		
I_s (MPa)	**0.95**	**0.89**	**0.70**		
$I_{s(50)}$ (MPa)	**0.93**	**0.90**	**0.70**		
Failure mode	Through fabric of specimen	Through fabric of specimen	Through fabric of specimen		

Notes:

Approved Signatory: C. Greely

Date: 16/11/2016

NATA
ACCREDITED FOR
TECHNICAL COMPETENCE

Accredited for compliance with ISO/IEC 17025.

NATA Accredited Laboratory Number: **17062**

11 Attachment F – Notes Relating To 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

Engineering reports are based on information that may be gained from limited subsurface site testing and sampling, supplemented by knowledge of local geology and experience. For this reason, they must be regarded as interpretative rather than factual documents, limited to some extent by the scope of information on which they rely.

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:

o 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.

o 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, structure, soil or rock type and inclusions.

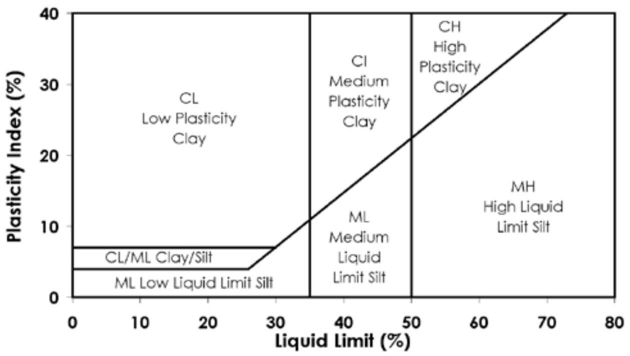
Particle Size

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	Size (mm)
BOULDERS		>200
COBBLES		63 to 200
GRAVEL	Coarse	20 to 63
	Medium	6 to 20
	Fine	2.36 to 6
SAND	Coarse	0.6 to 2.36
	Medium	0.2 to 0.6
	Fine	0.075 to 0.2
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.



Moisture Condition

Dry	Looks and feels dry. Cohesive and cemented soils are hard, friable or powdery. Uncemented granular soils run freely through hands.
Moist	Soil feels cool and damp and is darkened in colour. Cohesive soils can be moulded. Granular soils tend to cohere.
Wet	As for moist but with free water forming on hands when handled.

Consistency of Cohesive Soils

Cohesive soils refer to predominantly clay materials.

Term	C_u (kPa)	Approx. SPT "N"	Field Guide
Very Soft	<12	2	A finger can be pushed well into the soil with little effort. Sample extrudes between fingers when squeezed in fist.
Soft	12 - 25	2 – 4	A finger can be pushed into the soil to about 25mm depth. Easily moulded in fingers.
Firm	25 - 50	4 – 8	The soil can be indented about 5mm with the thumb, but not penetrated. Can be moulded by strong pressure in the figures.
Stiff	50 - 100	8 – 15	The surface of the soil can be indented with the thumb, but not penetrated. Cannot be moulded by fingers.
Very Stiff	100 - 200	15 – 30	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	> 200	> 30	The surface of the soil can be marked only with the thumbnail. Brittle. Tends to break into fragments.
Friable	-	-	Crumbles or powders when scraped by thumbnail.

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 - 35	5 - 10	2 - 5
Medium dense	35 - 65	10 - 30	5 - 15
Dense	65 - 85	30 - 50	15 - 25
Very dense	> 85	> 50	> 25

* Values may be subject to corrections for overburden pressures and equipment type.

Minor Components

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

Term	Assessment	Proportion of Minor component In:
Trace of	Presence just detectable by feel or eye. Soil properties little or no different to general properties of primary component.	Coarse grained soils: < 5 % Fine grained soils: < 15 %
With some	Presence easily detectable by feel or eye. Soil properties little different to general properties of primary component.	Coarse grained soils: 5 – 12 % Fine grained soils: 15 – 30 %

Symbols for Soils and Other

SOILS

▨ COBBLES/BOULDERS
▨ GRAVEL (GP OR GW)
▨ SILTY GRAVEL (GM)
▨ CLAYEY GRAVEL (GC)
▨ SAND (SP OR SW)
▨ SILTY SAND (SM)
▨ CLAYEY SAND (SC)

▨ SILT (ML OR MH)
▨ ORGANIC SILT (OH)
▨ CLAY (CL, CI OR CH)
▨ SILTY CLAY
▨ SANDY CLAY
▨ PEAT
▨ TOPSOIL

OTHER

▨ FILL
▨ TALUS
■ ASPHALT
▨ CONCRETE

Unified Soil Classification Scheme (USCS)

<table>
<tr><td colspan="6">FIELD IDENTIFICATION PROCEDURES
(Excluding particles larger than 63 mm and basing fractions on estimated mass)</td><td>USCS</td><td>Primary Name</td></tr>
<tr><td rowspan="8">COARSE GRAINED SOILS
More than 50 % of material less than 63 mm is larger than 0.075 mm</td><td rowspan="16">(A 0.075 mm particle is about the smallest particle visible to the naked eye)</td><td rowspan="4">GRAVELS
More than half of coarse fraction is larger than 2.0 mm.</td><td rowspan="2">CLEAN GRAVELS
(Little or no fines)</td><td colspan="2">Wide range in grain size and substantial amounts of all intermediate particle sizes.</td><td>GW</td><td>Gravel</td></tr>
<tr><td colspan="2">Predominantly one size or a range of sizes with more intermediate sizes missing</td><td>GP</td><td>Gravel</td></tr>
<tr><td rowspan="2">GRAVELS WITH FINES
(Appreciable amount of fines)</td><td colspan="2">Non-plastic fines (for identification procedures see ML below)</td><td>GM</td><td>Silty Gravel</td></tr>
<tr><td colspan="2">Plastic fines (for identification procedures see CL below)</td><td>GC</td><td>Clayey Gravel</td></tr>
<tr><td rowspan="4">SANDS
More than half of coarse fraction is smaller than 2.0 mm</td><td rowspan="2">CLEAN SANDS
(Little or no fines)</td><td colspan="2">Wide range in grain sizes and substantial amounts of intermediate sizes missing.</td><td>SW</td><td>Sand</td></tr>
<tr><td colspan="2">Predominantly one size or a range of sizes with some intermediate sizes missing</td><td>SP</td><td>Sand</td></tr>
<tr><td rowspan="2">SANDS WITH FINES
(Appreciable amount of fines)</td><td colspan="2">Non-plastic fines (for identification procedures see ML below)</td><td>SM</td><td>Silty Sand</td></tr>
<tr><td colspan="2">Plastic fines (for identification procedures see CL below)</td><td>SC</td><td>Clayey Sand</td></tr>
<tr><td rowspan="8">FINE GRAINED SOILS
More than 50 % of material less than 63 mm is smaller than 0.075 mm</td><td colspan="6">IDENTIFICATION PROCEDURES ON FRACTIONS < 0.2 MM</td></tr>
<tr><td>DRY STRENGTH (Crushing Characteristics)</td><td>DILATANCY</td><td>TOUGHNESS</td><td>DESCRIPTION</td><td>USCS</td><td>Primary Name</td></tr>
<tr><td>None to Low</td><td>Quick to Slow</td><td>None</td><td>Inorganic silts and very fine sands, rock flour, silty or clayey fine sands with slight plasticity</td><td>ML</td><td>Silt</td></tr>
<tr><td>Medium to High</td><td>None</td><td>Medium</td><td>Inorganic clays of low to medium plasticity ^{1}, gravelly clays, sandy clays, silty clays, lean clays</td><td>CL ^{2}</td><td>Clay</td></tr>
<tr><td>Low to Medium</td><td>Slow to Very Slow</td><td>Low</td><td>Organic slits and organic silty clays of low plasticity</td><td>OL</td><td>Organic Silt</td></tr>
<tr><td>Low to Medium</td><td>Slow to Very Slow</td><td>Low to Medium</td><td>Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts</td><td>MH</td><td>Silt</td></tr>
<tr><td>High</td><td>None</td><td>High</td><td>Inorganic clays of high plasticity, fat clays</td><td>CH</td><td>Clay</td></tr>
<tr><td>Medium to High</td><td>None</td><td>Low to Medium</td><td>Organic clays of medium to high plasticity</td><td>OH</td><td>Organic Silt</td></tr>
<tr><td colspan="2">HIGHLY ORGANIC SOILS</td><td colspan="4">Readily identified by colour, odour, spongy feel and frequently by fibrous texture</td><td>Pt</td><td>Peat</td></tr>
</table>

Notes:

1. Low Plasticity – Liquid Limit $W_L < 35$ % Medium Plasticity – Liquid limit W_L 35 to 60 % High Plasticity - Liquid limit $W_L > 60$ %.
2. CI may be adopted for clay of medium plasticity to distinguish from clay of low plasticity.

Soil Agricultural Classification Scheme

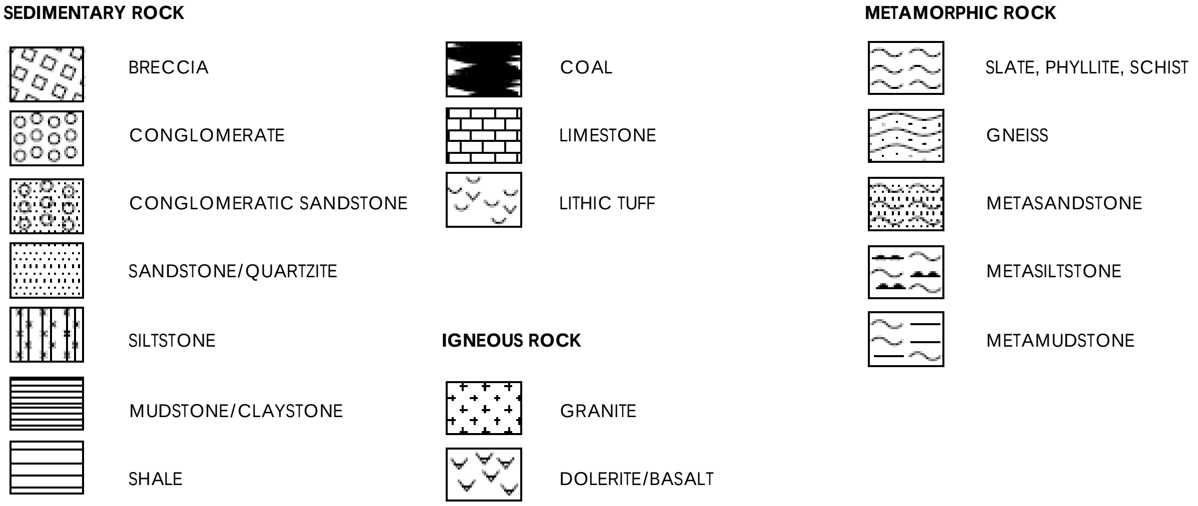
In some situations, such as where soils are to be used for effluent disposal purposes, soils are often more appropriately classified in terms of traditional agricultural classification schemes. Where a Martens report provides agricultural classifications, these are undertaken in accordance with descriptions 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

Explanation of Terms (1 of 2)

Symbols for Rock



Definitions

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

Rock Substance	In geotechnical engineering terms, rock substance is any naturally occurring aggregate of minerals and organic matter which cannot be disintegrated or remoulded by hand in air or water. Other material is described using soil descriptive terms. Rock substance is effectively homogeneous and may be isotropic or anisotropic.
Rock Defect	Discontinuity or break in the continuity of a substance or substances.
Rock Mass	Any body of material which is not effectively homogeneous. It can consist of two or more substances without defects, or one or more substances with one or more defects.

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	Soil derived from the weathering of rock. The mass structure and substance fabric are no longer evident. There is a large change in volume but the soil has not been significantly transported.
Extremely weathered[1]	EW	Rock substance affected by weathering to the extent that the rock exhibits soil properties - i.e. it can be remoulded and can be classified according to the Unified Classification System, but the texture of the original rock is still evident.
Highly weathered[2]	HW	Rock substance affected by weathering to the extent that limonite staining or bleaching affects the whole of the rock substance and other signs of chemical or physical decomposition are evident. Porosity and strength may be increased or decrease compared to the fresh rock usually as a result of iron leaching or deposition. The colour and strength of the original rock substance is no longer recognisable.
Moderately weathered[2]	MW	Rock substance affected by weathering to the extent that staining extends throughout the whole of the rock substance and the original colour of the fresh rock is no longer recognisable.
Slightly weathered	SW	Rock substance affected by weathering to the extent that partial staining or discolouration of the rock substance usually by limonite has taken place. The colour and texture of the fresh rock is recognisable.
Fresh	FR	Rock substance unaffected by weathering

Notes:
1 The term "Distinctly Weathered" (DW) may be used to cover the range of substance weathering between EW and SW.
2 Rs and EW material is described using soil descriptive terms.

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	Is (50) MPa	Field Guide	Symbol
Very low	>0.03 ≤0.1	May be crumbled in the hand. Sandstone is 'sugary' and friable.	VL
Low	>0.1 ≤0.3	A piece of 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	A piece of core 150mm long x 50mm diameter can be broken by hand with considerable difficulty. Readily scored with a knife.	M
High	>1 ≤3	A piece of core 150mm long x 50mm diameter cannot be broken by unaided hands, can be slightly scratched or scored with a knife.	H
Very high	>3 ≤10	A piece of core 150mm long x 50mm diameter may be broken readily with hand held hammer. Cannot be scratched with pen knife.	VH
Extremely high	>10	A piece of core 150mm long x 50mm diameter is difficult to break with hand held hammer. Rings when struck with a hammer.	EH

Explanation of Terms (2 of 2)

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)
●	Unconfined 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.

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:

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

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	◁	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 the rod weight only
HW	Penetration occurred under the hammer and rod weight only
HB 30/80mm	Hammer double bouncing on anvil after 80 mm penetration
N=18	Where practical refusal occurs, report blows and penetration for that interval

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		