Favelle Favco Cranes Pty Ltd
C/- Bureau SRH Pty Ltd

martens consulting engineers

Preliminary Geotechnical Assessment: Proposed Warehouse Development – 28 Yarrunga Street, Prestons, NSW

ENVIRONMENTAL

WATER

WASTEWATER

GEOTECHNICAL

CIVIL

PROJECT MANAGEMENT

P1907209JR02V02
November 2019

Limitations Statement

The sole purpose of this report and the associated services performed by Martens & Associates Pty Ltd is to complete a preliminary geotechnical assessment in accordance with the scope of services set out by Favelle Favco Cranes Pty Ltd C/- Bureau SRH Pty Ltd (hereafter known as the Client). That scope of works and services were defined by the requests of the Client, by the time and budgetary constraints imposed by the Client, and by the availability of access to the site.

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

Contents

1 Proposed Development and Investigation Scope

The proposed development details and investigation scope are summarised in Table 1.

Table 1: Summary of proposed development and investigation scope.

Item	Details
Property Address	28 Yarrunga Street, Prestons, NSW ('the site')
Lot/DP	Lot 2 in DP 536915
LGA	Liverpool City Council ('Council')
Assessment Purpose	Preliminary geotechnical assessment to support a Development Application (DA) and assist structural design of the proposed warehouse development and provision of a preliminary flexible pavement thickness design for proposed carparks.
Site Area	46839.7 m^2 (SJD, 2018)
Proposed Development	We understand from the proposal plans (BSRH, 2019) that the development will include: o Demolition of existing structures on site. o Construction of a new two storey warehouse with basement parking. This will likely require excavations between approximately 1.5 m below ground level (mbgl) in the southwest corner and 3.0 mbgl in the southeast corner. Proposed building setbacks are approximately 7.2 m and 7.5 m from the northern and southern boundaries, respectively, and approximately 16.8 m and 17.5 m from the eastern and western boundaries, respectively. Although, the carparks may be on a concrete slab, extending from warehouse floor slab, a flexible pavement option is also considered. Design advice for hardstands and heavy duty concrete pavements, required for trucks and cranes, has not been included in this report. Further advice can be provided, if required.
Investigation Scope of Work	o Review of DBYD survey plans and buried service locating. o A general site walkover survey. o Drilling of five boreholes (BH101 to BH105) up to investigation termination target depth of 4.1 metres below ground level (mbgl) (refer Attachment B for borehole logs, and associated explanatory notes in Attachment F). o Collection of soil and weathered rock samples for laboratory testing and future reference. o Five Dynamic Cone Penetrometer (DCP) tests (DCP101 to DCP105) up to 2.75 mbgl (refer DCP 'N' counts in Attachment C). Investigation locations are shown in Figure 1, Attachment A.
Laboratory Testing	Laboratory testing was carried out by Resource Laboratories, a National Association of Testing Authorities (NATA) accredited laboratory, on two soil samples to assess California Bearing Ratio (CBR).

2 General Site Details and Subsurface Conditions

General site details and investigation findings are summarised in Table 2.

Table 2: Summary of general site conditions based on desktop review, site walkover and site investigations.

Item	Comment
Topography	Within slightly undulating terrain, gently sloping towards the northwest
Typical Slopes, Aspect, Elevation	The site has a north westerly aspect with an overall grade of less than 5%. Site elevation ranges between approximately 42.4 mAHD (south eastern corner) and 34.9 mAHD (north western corner) (SJD, 2018).
Expected geology	Bringelly Shale consisting of shale, carbonaceous claystone, claystone, laminite, fine to medium-grained lithic sandstone, rare coal and tuff (*Penrith 1:100 000 Geological Sheet 9030,* 1st edition)
Existing Development	o Large metal building at the centre of the site, brick and metal building in the southwest corner of the site, and small metal buildings scattered throughout the site. o Crane bases are located mainly in the southern portion of the site.
Vegetation	Grass, shrubs and scattered trees particularly along the southern boundary of the site.
Drainage	Via overland flow and onsite drainage via pit and pipe towards the northwest network.
Neighbouring environment	Single storey fibro commercial building to the west and single and double storey metal commercial buildings to the east and north, respectively.
Sub-surface Soil / Rock Units	Investigation revealed the following generalised subsurface units likely underlie the site: Unit A: Topsoil comprising very stiff dry silty clay encountered from ground surface up to approximately 0.2 mbgl (BH101). Unit B: Fill comprising dry silty clay encountered in the northern portion of the site from ground surface up to approximately 1.0 mbgl (BH104 and BH105). Fill has likely been placed for previous site development and / or levelling purposes and assumed to have been placed under 'uncontrolled' conditions due to absence of quality control documentation and presence of unsuitable materials. The extent of filling was not determined. Unit C: Residual soil comprising stiff to hard clay, of between 0.7 m and 2.5 m thickness, encountered up to approximately 3.0 mbgl. Unit D: Extremely to highly weathered and inferred extremely low to very low strength siltstone and shale from depths of between 1.0 mbgl and 3.0 mbgl, grading to very low to low strength encountered up to investigation termination depth of 4.1 mbgl. We assume similar conditions below 4.1 mbgl, with the presence of possible higher and lower strength bands.
Groundwater	Groundwater inflow was not encountered during drilling of the boreholes up to 4.1 mbgl. Groundwater may be encountered within excavations, however a very low inflow rate would be expected. Ephemeral perched groundwater may be encountered within the soil profile and / or at the soil / rock interface at times of, and following, heavy or extended periods of rainfall.

3 Preliminary Pavement Thickness Design

3.1 Overview

Preliminary flexible pavement thickness design for the proposed carparks was undertaken in accordance with Liverpool City Council's Development Design Specification D2 – Pavement Design (2003) and Austroads (2017) - Guide to Pavement Technology (Part 2).

3.2 Design Parameters

Equivalent Standard Axles (ESA)

A traffic loading of 1.5 x 10^5 Equivalent Standard Axles (ESA) should be adopted in accordance with Austroads (2017).

3.3 CBR Test Results

Two bulk soil samples were collected from BH101 and BH103 and submitted to Resources Laboratory, a National Association of Testing Authority (NATA) accredited laboratory for CBR testing. A four day soaked CBR test was conducted in accordance with AS1289.1.1, 2.1.1, 5.1.1 and 6.1.1. Test results are summarised in Table 3. A laboratory test certificate is provided in Attachment D.

Table 3: CBR test results.

Borehole Number	Sample Depth (mbgl)	Material	CBR [1] Value (%)
BH101	0.3 – 0.7	Clay	1.5
BH103	0.2 – 0.6	Silty Clay	6

Notes:

1. Four day soak, compacted to 98 % SMDD (±2 % of OMC), applying a 4.5 kg surcharge.

Based on the findings of the geotechnical investigation, the proposed northwest carpark is underlain by up to 0.5 m of uncontrolled fill (BH105) comprising hard silty clay of medium to high plasticity. The proposed southwest carpark is underlain by residual clay (BH103) comprising very stiff silty clay of high plasticity. CBR values of 1.5% and 6% were obtained for the residual clay subgrade likely to be encountered at the proposed carpark locations.

Subgrade improvement / replacement will be required considering a CBR of <3% was obtained from testing of a representative subgrade material, and where material of inferior quality or uncontrolled fill is uncovered during excavation. Subject to subgrade preparation works having been carried out (refer Section 3.5.1), a CBR value of 3% may be adopted for preliminary design purposes.

Additional CBR testing may be carried out to provide a better indication of subgrade conditions across the car park locations and hardstand locations, considering final design levels and subgrade preparation results. The additional testing may be undertaken following the issue of Construction Certification.

3.4 Pavement Thickness

Table 4 presents recommended pavement materials and material thicknesses for the proposed carparks.

Table 4: Preliminary pavement material thickness design for CBR 3 % for carpark.

Pavement Type	Total Thickness (mm)	Layer	Thickness (mm)	Materials
Carpark	425 (including wearing course)	Wearing Course	25	25 mm Asphalt Concrete (AC10) without seal
		Base	150 [1]	DGB20
		Sub-base	250 [1]	DGS20 or DGS40

Notes:

1. Based on Figure 8.4 of AUSTROADS AGPT02 (2017).

3.5 Earthworks

3.5.1 Subgrade Preparation

The subgrade is to be trimmed and compacted, following the removal of topsoil and other unsuitable materials such as root containing soils and uncontrolled fill. Treat at least 300 mm depth subgrade by:

- Removal and replacement with approved granular material under geotechnical engineer's direction. Disposal of excavated material off site should be carried out in accordance with NSW DECC (2009) Waste Classification Guidelines.

- In-situ stabilisation of residual soil with cement / lime or similar binding agent. It is envisaged that approximately 3 to 5 % of lime or similar binding agent will be required, subject to confirmation by laboratory testing of selective soil samples and binding agent.

- Mixing of granular material with the residual soil to increase its CBR value.

Density testing of the upper 300 mm layer should be carried out at a minimum rate of one test per 500 m^3 distributed reasonably evenly throughout full depth and area (refer Table 8.1 of AS3798). Minimum relative density of subgrade shall be 100 % Maximum Dry Density (MDD) at a standard compactive effort within 2 % of optimum moisture content (OMC).

3.5.2 Subsoil Drainage

Surface and sub-soil drainage is to be provided in accordance with Liverpool City Council requirements.

3.5.3 Placement and Testing of Pavement Material

Pavement materials shall be placed in layers (when compacted) not thicker than 200 mm or less than 100 mm. Pavement materials shall be compacted to the following condition:

- Sub-base - Minimum 98 % MDD at modified compactive effort (±2% OMC).
- Base - Minimum 98% MDD at modified compactive effort (±2% OMC).

Compaction testing shall be undertaken by a NATA accredited laboratory in accordance with procedures as outlined in Liverpool City Council (2003) Development Design Specification D2 – Pavement Design. Each pavement layer shall be proof rolled under Geotechnical Engineers' supervision. Subsequent pavement layers shall not be placed prior to approval of underlying layer by the Geotechnical Engineer.

3.6 Fill Placement

Where filling is required to raise subgrade levels, the use of site-won excavated residual soils may be considered, subject to treatment with lime / gypsum / granular material. Alternatively, suitable granular fill, approved for use by a Geotechnical Engineer may be placed. All earthworks specifications, including additional testing requirements such as CBR testing, are to be prepared by the supervising engineer and be implemented by the contractor.

3.7 General Recommendations

Presented specifications are to, as a minimum, address the requirements of AS 3798 (2007) and Liverpool City Council's Development Design Specification D2 - Pavement Design.

4 Geotechnical Assessment

4.1 Preliminary Soil and Rock Properties

Preliminary soil and rock properties inferred from observations during borehole drilling, such as auger penetration resistance, DCP test results as well as engineering assumptions are summarised in Table 5.

Table 5: Preliminary material properties.

Layer	$\gamma_{in\text{-}situ}$ [2] (kN/m³)	C_u [3] (kPa)	C' (kPa)	Ø' [4] (deg)	E' [5] (MPa)	K_s [6] (MPa/m)	ABC [7]
TOPSOIL / FILL [1]: Silty CLAY	17	NA [8]	NA [8]	NA [8]	NA [8]	NA [8]	NA [8]
RESIDUAL: CLAY (stiff to very stiff)	18	100	5	28	20	20	150
RESIDUAL: CLAY (hard)	19	200	6	30	45	45	250
WEATHERED ROCK: SILTSTONE / SHALE (extremely low to very low strength)	22	NA [8]	20	28	200	50	350
WEATHERED ROCK: SHALE (very low to low strength)	23	NA [8]	50	28	300	100	400

Notes:

1. Assumed 'uncontrolled' fil and variable in depth across the site.
2. Material in-situ unit weight, based on visual assessment (±10 %).
3. Undrained shear strength (± 5 kPa) estimate assuming normally consolidated clay in a dry condition.
4. Average effective internal friction angle (±2 °) estimate assuming drained conditions.
5. Effective elastic modulus (±10 %) estimate.
6. Modulus of subgrade reaction estimate. For horizontal modulus, 1/3 vertical Ks may be adopted.
7. Allowable end bearing capacity (kPa) for shallow footings embedded at least 0.3, subject to confirmation on site by a geotechnical engineer of inferred foundation conditions.
8. Not applicable, or not recommended either due to depth or potential internal settlement of materials.

4.2 Recommendations

Geotechnical recommendations for site development are provided below. Further general geotechnical recommendations are provided in Attachment E.

1. Excavation Support: Excavations greater than 1 m need to be temporarily and permanently battered back, to maintain excavation stability and limit potential adverse impacts on

surrounding structures / neighbouring properties. All excavation work should be completed with reference to the most recent version of Code of Practice 'Excavation Work', by Safe Work Australia. Typically, the building setback from the site boundary is higher than 7.0 m, giving sufficient space to batter the slope safely from the building setback wall to property boundaries. Proposed excavations for the development are not expected to extend into the zone of influence of neighbouring properties.

As such, excavation may be temporarily battered back at 1 V : 2 H for stiff to very stiff silty clay / clay and 1 V : 1.5 H for hard clay and for extremely to highly weathered, extremely low to very low strength siltstone and shale. Permanent batters should not exceed 1 V : 3 H. These recommendations are subject to inspection and approval by a geotechnical engineer on site. All earthworks should be carried out in accordance with AS3798 (2007). A qualified geotechnical engineer should inspect and approve earthworks.

Alternatively, the excavation may be retained by cantilevered soldier piles with shotcrete infill panels. The soldier pile wall may be included in permanent retaining wall design. Pile lengths may be reduced where building floor slabs act as permanent lateral support for piles. Tie-back anchors may also be considered to minimise pile length, subject to the available space and permission from neighbouring land owners for anchor installation.

2. Earth Pressure Coefficients: Shoring or retaining wall design, may adopt preliminary active, at rest and passive earth pressure coefficients of:

    - 0.39, 0.56, 2.56 for stiff to very stiff residual clay.

    - 0.33, 0.50, 3.00 for hard residual clay.

    These recommendations are subject to level retained ground behind the retaining wall, drainage provisions behind the wall and no structural loading impacts.

    Shoring / retaining wall design should also accommodate pressures imposed by a rock wedge with a failure plane extending at 45° away from excavation base level up to top of rock.

3. Footings and Foundations: It is expected that shallow footings, such as pad and strip footings, or slab-on-ground is likely to be adopted as support for the new warehouse. Individual pad

footings and all footings within the building footprint should not span the interface between different foundation materials. Alternatively, inclusion of movement joints may mitigate impacts of differential ground movements between reactive clay and rock foundation materials.

Allowable bearing capacities for shallow footings founding in residual soil and weathered rock are provided in Table 5.

Should higher bearing capacities be required, further assessment will be required.

4. Drainage requirements: Appropriate surface and sub-surface drainage should be provided to divert overland flows and collected groundwater, away from excavations, retaining walls or foundations and limit ponding of water in excavations or near footings. Collected water should be discharged into council approved stormwater systems downslope of the site.

   Ephemeral perched groundwater inflow, if encountered during excavation of rock, is expected to be limited and can be managed by sump and pump methods.

5. Site Classification: The site is classified as a "H1" site in accordance with AS 2870 (2011), to be confirmed by lab testing. This classification is subject to recommendations presented in this report and design of footings in accordance with the relevant Australian standards and guidelines.

5 Proposed Additional Works

5.1 Works Prior to Construction Certificate

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

1. Drill additional boreholes to confirm our findings, including rock coring and point load testing of collected rock samples to assess rock strength if higher end bearing pressures are required.

2. Carry out further laboratory testing for Atterberg limits and linear shrinkage to determine the shrinkage limit, plastic limit, and liquid limit of the clay layers. Additional CBR testing should be carried out to gain a better understanding of the subgrade condition and to assist design of hardstands, if required.

3. Determine extent of filling across the site, as necessary, and implications on the proposed development.

4. Review by a senior geotechnical engineer of the final foundation designs, if not carried out by MA, to confirm adequate consideration of the geotechnical risks and adoption of the recommendations provided in this report.

5.2 Construction Monitoring and Inspections

We recommend the following is inspected and monitored during construction of the project (Table 6).

Table 6: Recommended inspection / monitoring requirements during site works.

Scope of Works	Frequency/Duration	Who to Complete
Inspect temporary excavation batters. Monitor associated performance to assess need for additional support requirements.	Daily / As required [2]	Builder / MA [1]
Monitor groundwater seepage from excavation slopes, if encountered, to assess stability of exposed materials and need for additional drainage requirements.	When encountered	Builder / MA [1]
Inspect exposed material at foundation / subgrade level to verify suitability as foundation / lateral support / subgrade.	Prior to reinforcement set-up and concrete placement, or fill placement	MA [1]
Monitor excavation-induced vibrations if excavation of medium or higher strength rock by rock hammer is required.	Daily at on-set of excavation and as agreed thereafter [2]	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

Austroads Ltd (2017) Guide to Pavement Technology, Part 2: Pavement Structural Design, Publication No. AGPT02-12, ISBN 978-1-921991-11-0.

Bureau SRH Architecture (2019) *Architectural Drawings*, Project No. 18144, Drawing Nos. DD301 and DD302, dated April 2019. (BSRH, 2019)

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Liverpool City Council (2003) Development Design Specification D2 - Pavement Design.

S.J. Dixon Surveyors (2018) Detail and Level Plan, Reference No. 51995, Revision A, dated October 2018. (SJD, 2018)

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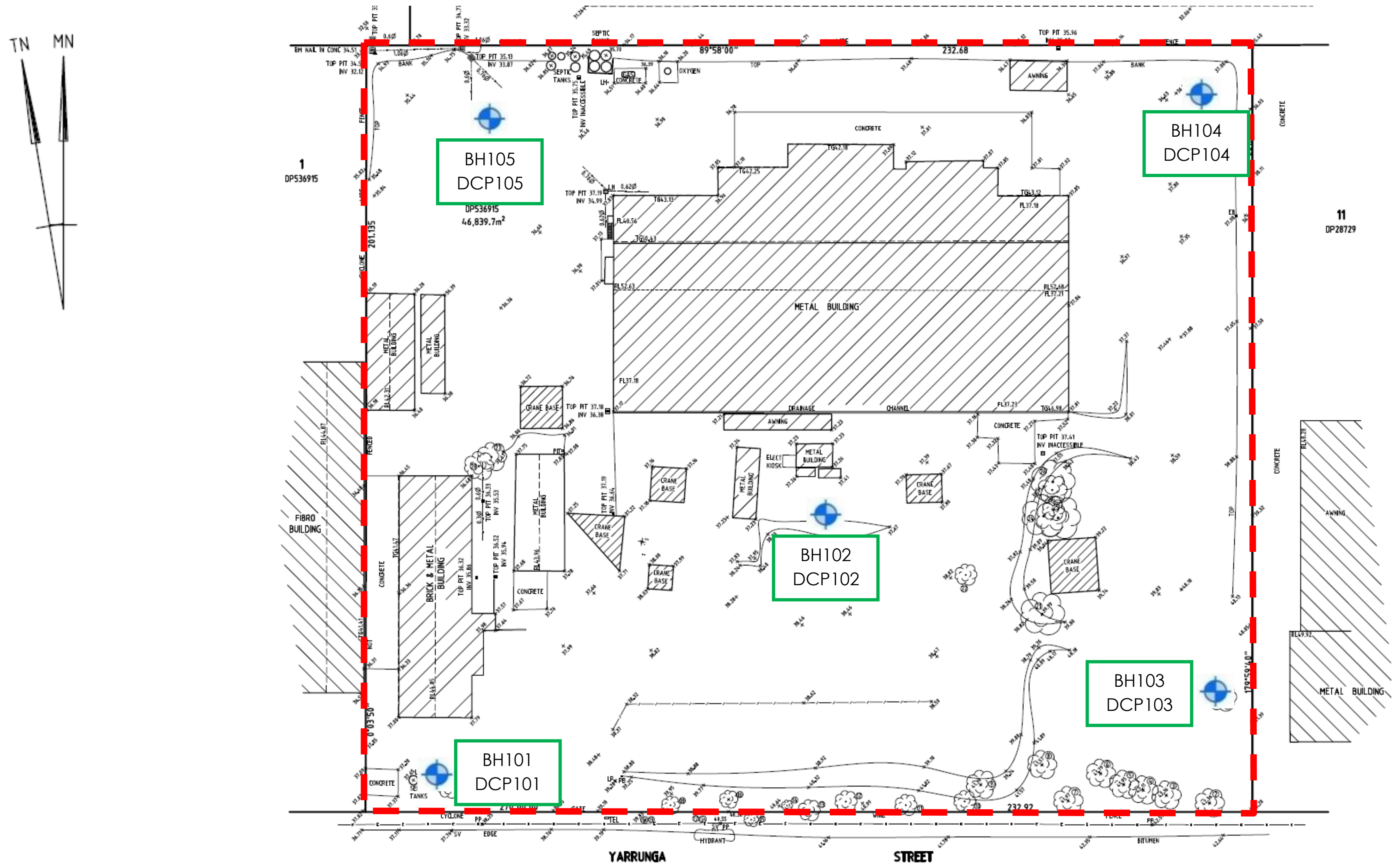
Standards Australia Limited (2017) AS 1726:2017, *Geotechnical site investigations*, SAI Global Limited.

Standards Australia Limited (1993) AS 2187.2:1993, *Explosives—Storage, transport and use*, Part 2: Use of explosives, SAI Global Limited.

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

Standards Australia Limited (2007) AS 3798:2007, *Guidelines on earthworks for commercial and residential developments*, SAI Global Limited.

7 Attachment A – Site layout and Geotechnical Testing Plan



martens

Key:

Approximate borehole and DCP test location

Indicative site boundary

Martens & Associates Pty Ltd ABN 85 070 240 890		Environment | Water | Wastewater | Geotechnical | Civil | Management	
Drawn:	CGL	**SITE LAYOUT AND GEOTECHNICAL TESTING PLAN** **28 Yarrunga Street, Prestons, NSW** (Source: SJD, 2018)	Drawing No: **FIGURE 1**
Approved:	SK		
Date:	08.08.2019		
Scale:	NA		Project No: P1907209JR02V02

8 Attachment B – Test Borehole Logs

CLIENT	Favelle Favco Cranes Pty Ltd	COMMENCED	18/07/2019	COMPLETED	18/07/2019	**REF BH101**	
PROJECT	Preliminary Geotechnical Assessment	LOGGED	CGL	CHECKED	SK	Sheet 1 OF 1	
SITE	28 Yarrunga St, Prestons, NSW	GEOLOGY	Bringelly Shale	VEGETATION	Grass	PROJECT NO. P1907209	
EQUIPMENT	4WD truck-mounted hydraulic drill rig	EASTING		RL SURFACE	37.6 m	DATUM	AHD
EXCAVATION DIMENSIONS	Ø100 mm x 4.10 m depth	NORTHING		ASPECT	Northwest	SLOPE	<5%

Drilling | **Sampling** | **Field Material Description**

METHOD	PENETRATION RESISTANCE	WATER	DEPTH (metres)	DEPTH / RL	SAMPLE OR FIELD TEST	USCS / ASCS CLASSIFICATION	SOIL/ROCK MATERIAL DESCRIPTION	MOISTURE CONDITION	CONSISTENCY DENSITY	STRUCTURE AND ADDITIONAL OBSERVATIONS
AD/V	L-M	Not Encountered	0.00–0.20	37.60		CI-CH	TOPSOIL: Silty CLAY: medium to high plasticity; dark brown; trace rootlets; trace sand.	M (<<PL)		TOPSOIL
AD/V	L-M / H	Not Encountered	0.20–2.20	0.20 / 37.40	0.3/S/1 D 0.30 m 0.3-0.7/CBR/1 D 0.30-0.70 m 0.8/S/1 D 0.80 m 1.2/S/1 D 1.20 m 1.5/S/1 D 1.50 m	CH	CLAY: high plasticity; red, grey and brown; with silt; trace fine subangular gravels.	M (<<PL)	H / VSt / H	RESIDUAL SOIL 1.50: Possible extremely weathered rock.
AD/T	L	Not Encountered	2.20–3.30	2.20 / 35.40	2.3/R/1 D 2.30 m		SHALE: yellow-brown; inferred extremely low strength; extremely weathered.			WEATHERED ROCK 2.20: V-bit refusal.
AD/T	M		3.30–3.70	3.30 / 34.30			Inferred extremely low to very low strength; extremely to highly weathered.			
AD/T	H		3.70–4.10	3.70 / 33.90	3.8/R/1 D 3.80 m		Inferred very low to low strength; highly weathered.			
			4.10	4.10			Hole Terminated at 4.10 m (Target depth reached)			

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

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Engineering Log - BOREHOLE

CLIENT	Favelle Favco Cranes Pty Ltd	COMMENCED	18/07/2019	COMPLETED	18/07/2019	**REF BH102**
PROJECT	Preliminary Geotechnical Assessment	LOGGED	CGL	CHECKED	SK	Sheet 1 OF 1
SITE	28 Yarrunga St, Prestons, NSW	GEOLOGY	Bringelly Shale	VEGETATION	None	PROJECT NO. P1907209
EQUIPMENT	4WD truck-mounted hydraulic drill rig	EASTING		RL SURFACE	37.4 m	DATUM: AHD
EXCAVATION DIMENSIONS	Ø100 mm x 4.00 m depth	NORTHING		ASPECT	Northwest	SLOPE: <5%

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
AD/V	L	Not Encountered		37.40	0.2/S/1 D 0.20 m 0.6/S/1 D 0.60 m			CH	CLAY: high plasticity; red, white and brown; trace silt.	M (<PL)	F - St	RESIDUAL SOIL
	M										VSt	
	H			0.80 / 36.60					Yellow, brown and white; with fine subangular gravels.	M (<<PL)	H	
AD/T	M		1	1.00 / 36.40	1.2/R/1 D 1.20 m 1.8/R/1 D 1.80 m 2.4/R/1 D 2.40 m				SHALE: yellow-brown; inferred extremely low to very low strength; extremely to highly weathered; with clay bands.			WEATHERED ROCK 1.00: V-bit refusal.
			2									
			3									
	H			3.50 / 33.90					Inferred very low to low strength; highly weathered.			
			4	4.00					Hole Terminated at 4.00 m (Target depth reached)			
			5									

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Engineering Log - BOREHOLE

CLIENT	Favelle Favco Cranes Pty Ltd	COMMENCED	18/07/2019	COMPLETED	18/07/2019	REF BH103
PROJECT	Preliminary Geotechnical Assessment	LOGGED	CGL	CHECKED	SK	Sheet 1 OF 1
SITE	28 Yarrunga St, Prestons, NSW	GEOLOGY	Bringelly Shale	VEGETATION	None	PROJECT NO. P1907209

EQUIPMENT	4WD truck-mounted hydraulic drill rig	EASTING		RL SURFACE	41.2 m	DATUM	AHD
EXCAVATION DIMENSIONS	Ø100 mm x 2.50 m depth	NORTHING		ASPECT	Northwest	SLOPE	<5%

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
AD/V	M	Not Encountered		41.20	0.2/S/1 D 0.20 m 0.2-0.6/CBR/1 D 0.20-0.60 m			CH	Silty CLAY: high plasticity; red, grey and brown; with sand; with fine subangular gravels.	M (<<PL)	VSt	RESIDUAL SOIL
AD/V	H	Not Encountered	1	0.80 40.40 1.00 40.20	1.3/S/1 D 1.30 m			CH	CLAY: high plasticity; yellow-brown; with sand; with silt; with fine subangular gravels. Pale brown.		H	
AD/T	M		2	1.50 39.70 2.50	1.7/R/1 D 1.70 m				SILTSTONE: pale brown; inferred extremely low to very low strength; extremely to highly weathered.			WEATHERED ROCK 1.50: V-bit refusal.
			3 4 5						Hole Terminated at 2.50 m (Target depth reached)			

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

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Engineering Log - BOREHOLE

CLIENT	Favelle Favco Cranes Pty Ltd	COMMENCED	18/07/2019	COMPLETED	18/07/2019	REF BH104
PROJECT	Preliminary Geotechnical Assessment	LOGGED	CGL	CHECKED	SK	Sheet 1 OF 1
SITE	28 Yarrunga St, Prestons, NSW	GEOLOGY	Bringelly Shale	VEGETATION	None	PROJECT NO. P1907209
EQUIPMENT	4WD truck-mounted hydraulic drill rig	EASTING		RL SURFACE	36.9 m	DATUM: AHD
EXCAVATION DIMENSIONS	Ø100 mm x 4.00 m depth	NORTHING		ASPECT	Northwest	SLOPE: <5%

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
AD/V	H	Not Encountered		36.90	0.5/S/1 D 0.50 m			CI-CH	FILL: Silty CLAY: medium to high plasticity; dark brown and black; with fine to medium subangular gravels; inferred hard.		H	FILL
	L		1	1.00 / 35.90	1.2/S/1 D 1.20 m 1.2/S/2 D 1.20 m			CH	CLAY: high plasticity; red, white and brown; trace silt; trace fine subangular gravels.	M (<<PL)	St - VSt	RESIDUAL SOIL
				1.50 / 35.40	1.8/S/1 D 1.80 m				Pale brown.			
	H		2	2.00 / 34.90	2.3/S/1 D 2.30 m				Red, brown and white.		H	2.40: Possible extremely weathered rock.
AD/T	L		3	3.00 / 33.90					SHALE; inferred extremely low strength; extremely to highly weathered.			WEATHERED ROCK 3.00: V-bit refusal.
	M			3.30 / 33.60	3.8/R/1 D 3.80 m				Inferred extremely low to very low strength; highly weathered.			
			4	4.00					Hole Terminated at 4.00 m (Target depth reached)			
			5									

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

MARTENS & ASSOCIATES PTY LTD
Suite 201, 20 George St. Hornsby, NSW 2077 Australia
Phone: (02) 9476 9999 Fax: (02) 9476 8767
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Engineering Log - BOREHOLE

CLIENT	Favelle Favco Cranes Pty Ltd	COMMENCED	18/07/2019	COMPLETED	18/07/2019	REF BH105
PROJECT	Preliminary Geotechnical Assessment	LOGGED	CGL	CHECKED	SK	Sheet 1 OF 1
SITE	28 Yarrunga St, Prestons, NSW	GEOLOGY	Bringelly Shale	VEGETATION	None	PROJECT NO. P1907209
EQUIPMENT	4WD truck-mounted hydraulic drill rig	EASTING		RL SURFACE	36 m	DATUM: AHD
EXCAVATION DIMENSIONS	Ø100 mm x 4.00 m depth	NORTHING		ASPECT	Northwest	SLOPE: <5%

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
AD/V	H	Not Encountered		36.00	0.2/S/1 D 0.20 m			CI-CH	FILL: Silty CLAY: medium to high plasticity; grey; with fine subangular gravels; inferred hard.		H	FILL 0.20: Trace metal sheets.
AD/V	L	Not Encountered	1	0.50 35.50	1.0/S/1 D 1.00 m			CH	CLAY: high plasticity; orange, brown and white; trace silt; trace fine subangular gravels.	M (<<PL)	St - VSt	RESIDUAL SOIL
AD/V	L	Not Encountered	2	1.50 34.50	1.6/S/1 D 1.60 m				No gravels.	M (<PL) M (=PL)	VSt - H	
AD/V	H	Not Encountered			2.5/S/1 D 2.50 m					M (<PL)	H	2.50: Possible extremely weathered rock.
AD/T	M	Not Encountered	3	3.00 33.00	3.8/R/1 D 3.80 m				SHALE: Inferred extremely low to very low strength; extremely to highly weathered.			WEATHERED ROCK 3.00: V-bit refusal.
			4	4.00					Hole Terminated at 4.00 m (Target depth reached)			
			5									

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

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Engineering Log - BOREHOLE

9 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.au

Site	28 Yarrunga Street, Prestons, NSW	**DCP Group Reference**	P1907209JS01V01
Client	Favelle Favco Cranes Pty Ltd	**Log Date**	18/07/2019
Logged by	CGL		
Checked by	SK		
Comments	DCP commenced 50 mm below ground level.		

TEST DATA

Depth Interval (m)	DCP101	DCP102	DCP103	DCP104	DCP105			
0.15	9	3	25	Started @ 0.95 mbgl due to gravels in FILL	Started @ 0.5 mbgl due to gravels in FILL			
0.30	19	2	16					
0.45	19	3	9					
0.60	12	4	10		6			
0.75	11	11	15		7			
0.90	10	30	14		10			
1.05	11	Terminated due to bounce @ 0.95 mBGL	23	10	6			
1.20	14		17	7	6			
1.35	15		18	7	5			
1.50	37/120mm		22	7	6			
1.65	Terminated due to bounce @ 1.52 mBGL		Terminated due to bounce @ 1.55 mBGL	6	5			
1.80				6	7			
1.95				5	7			
2.10				8	14			
2.25				30	16			
2.40				20/80mm	24			
2.55				Terminated due to bounce @ 2.38 mBGL	27			
2.70					35			
2.85					Terminated due to high 'N' counts @ 2.75 mBGL			
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								

10 Attachment D – Laboratory Test Certificate

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
Job number: 19-0076
Project: P1907209
Report number: 1
Location: 28 Yarrunga Street, Prestons, NSW

California Bearing Ratio

Sampling method: Tested as received
Test method(s): AS 1289.1.1, 2.1.1, 5.1.1, 6.1.1

	Results			
Laboratory sample no.	19579	19580		
Customer sample no.	7209/BH101/ 0.3-0.7/S/1	7209/BH103/ 0.2-0.6/S/1		
Date sampled	18/07/2019	18/07/2019		
Material description	CLAY, with silt, trace of gravel, mottled red/ grey/brown	silty CLAY, with gravel and sand, mottled red/ brown/grey		
Maximum dry density (t/m^3)	1.71	1.72		
Optimum moisture content (%)	19.8	18.5		
Field moisture content (%)	n/a	n/a		
Oversize retained on 19.0mm sieve (%)	0	1		
Minimum curing time (hours)	48	48		
Dry density before soak (t/m^3)	1.68	1.70		
Dry density after soak (t/m^3)	1.63	1.68		
Moisture content before soak (%)	19.6	18.3		
Moisture content after soak (%)	23.4	21.5		
Moisture content after test - top 30mm (%)	28.2	22.0		
Moisture content after test - remaining depth (%)	21.4	19.9		
Density ratio before soaking (%)	98.5	98.5		
Moisture ratio before soaking (%)	99.0	99.0		
Period of soaking (days)	4	4		
Compactive effort	Standard	Standard		
Mass of surcharge applied (kg)	4.5	4.5		
Swell after soaking (%)	3.5	1.0		
Penetration (mm)	2.5	2.5		
CBR Value (%)	**1.5**	**6**		

Notes: Specified LDR: 98 ±1%

Method of establishing plasticity level - Visual / tactile

Approved Signatory: E. Maldonado **Date:** 19/08/2019

NATA
ACCREDITED FOR TECHNICAL COMPETENCE

Accredited for compliance with ISO/IEC 17025 - Testing.

NATA Accredited Laboratory Number: **17062**

11 Attachment E – General Geotechnical Recommendations

Geotechnical Recommendations

Important Recommendations About Your Site (1 of 2)

martens consulting engineers

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.

12 Attachment F – 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.

- o Changes in guidelines, standards and policy or interpretation of guidelines, standards and policy by statutory authorities.

- o The actions of contractors responding to commercial pressures.

- o 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.

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.

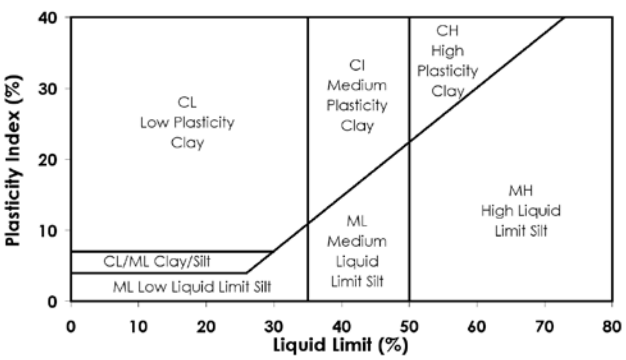
Particle Size

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		OTHER
COBBLES/BOULDERS	SILT (ML OR MH)	FILL
GRAVEL (GP OR GW)	ORGANIC SILT (OH)	TALUS
SILTY GRAVEL (GM)	CLAY (CL, CI OR CH)	ASPHALT
CLAYEY GRAVEL (GC)	SILTY CLAY	CONCRETE
SAND (SP OR SW)	SANDY CLAY	
SILTY SAND (SM)	PEAT	
CLAYEY SAND (SC)	TOPSOIL	

Unified Soil Classification Scheme (USCS)

<table>
<tr><th colspan="6">FIELD IDENTIFICATION PROCEDURES
(Excluding particles larger than 63 mm and basing fractions on estimated mass)</th><th>USCS</th><th>Primary Name</th></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>
<tr><td colspan="8">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.</td></tr>
</table>

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Soil Agricultural Classification Scheme

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

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

Rock Data

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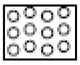
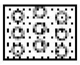
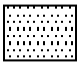
Symbols for Rock

SEDIMENTARY ROCK

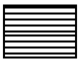
 BRECCIA

 CONGLOMERATE

 CONGLOMERATIC SANDSTONE

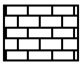
 SANDSTONE/QUARTZITE

 SILTSTONE

 MUDSTONE/CLAYSTONE

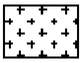
 SHALE

 COAL

 LIMESTONE

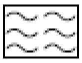
 LITHIC TUFF

IGNEOUS ROCK

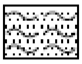
 GRANITE

 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 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 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	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

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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:

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

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

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

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

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

Standard Penetration Testing (SPT)

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

The test procedure is described in AS 1289.6.3.1-2004. The test is carried out in a borehole by driving a 50 mm diameter split sample tube under the impact of a 63 kg hammer with a free fall of 760 mm. It is normal for the tube to be driven in three successive 150 mm penetration depth increments and the 'N' value is taken as the number of blows for the last two 150 mm depth increments (300 mm total penetration). In dense sands, very hard clays or weak rock, the full 450 mm penetration may not be practicable and the test is discontinued. The test results are reported in the following form:

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

as 4, 6, 7
N = 13

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

as 15, 30/40 mm.

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

Dynamic Cone (Hand) Penetrometers

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

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

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

Pocket Penetrometers

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

$q_u = 2 \times C_u$.

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

Test Pit / Borehole Logs

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

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

Laboratory Testing

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

Ground Water

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

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

More reliable measurements can be made by installing standpipes, which are read at intervals over several days, or perhaps weeks for low permeability soils. Piezometers sealed in a particular stratum, may be advisable in low permeability soils or where there may be interference from a perched water table.

Test, Drill and Excavation Methods

Explanation of Terms (3 of 3)

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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 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		