Opal Aged Care C/- Pact PM

martens consulting engineers

Geotechnical Assessment: Proposed Opal Aged Care Facility 94-100 Explorers Way, St Clair, NSW

ENVIRONMENTAL

WATER

WASTEWATER

GEOTECHNICAL

CIVIL

PROJECT MANAGEMENT

P2007910JR02V01
March 2021

Limitations Statement

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Martens & Associates Pty Ltd derived the data in this report primarily from a number of sources including site inspections, correspondence regarding the proposal, examination of records in the public domain, interviews with individuals with information about the site or the project, and field explorations conducted on the dates indicated. The passage of time, manifestation of latent conditions or impacts of future events may require further examination / exploration of the site and subsequent data analyses, together with a re-evaluation of the findings, observations and conclusions expressed in this report.

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Document and Distribution Status							
Author(s)		**Reviewer(s)**		**Project Manager**		**Signature**	
Wasiul Bari		Kenneth Burgess		Jeff Fulton			
				Document Location			
Revision No.	**Description**	**Status**	**Release Date**	File Copy	Opal Aged Care c/- Pact PM		
1	Geotechnical Assessment	Draft	05.11.2020	1E, 1H, 1P	1P		
1	Geotechnical Assessment	Final	09.03.2021	1E, 1H, 1P	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 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	94-100 Explorers Way, St Clair, NSW ('the site')
Lot/DP	Lot 36 in DP 239502
LGA	Penrith City Council ('Council')
Assessment Purpose	Geotechnical assessment to support a Development Application (DA) and assist preliminary structural design of the proposed development.
Site Area	Approximately 1,050 m^2 (based on SIX Maps)
Proposed Development	Based on client provided information, the proposed development will include: o Demolition of existing site structures. o Construction of a new at-grade two storey aged care facility and associated car parking. Limited excavation or filling (i.e. less than 1 m) is expected to be required as part of construction works.
Previous Assessment	A geotechnical investigation was previously undertaken by Ground Engineering Design Pty Ltd in 2015 to support a residential subdivision. The investigation involved excavation of three test pits (TP1 to TP3) as shown in Figure 1, Attachment A. The findings and recommendations of the geotechnical investigation are presented in Ground Engineering Design's report referenced 6654725, dated June 2015 (GED, 2017). Results have not been reproduced in this report unless integral to our assessment.
Investigation Scope of Work	Field investigations conducted on 6 October 2020 included: o Review of DBYD survey plans and buried service search. o General site walkover to gain an appreciation of the site. o Seven boreholes (BH101 to BH107) up to 6.6 m below ground level (mbgl) via solid flight auger. 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 for future reference. o Six Dynamic Cone Penetrometer (DCP) tests (DCP101 to DCP106) up to 3.35 mbgl (refer DCP 'N' counts in Attachment C). Investigation locations are shown in Figure 1, Attachment A.
Laboratory Testing	Testing carried out by National Association of Testing Authorities (NATA) accredited laboratories included: o California bearing ratio (CBR) testing on two bulk soil sample by Resource Laboratories. o Salinity and exposure classification testing on six soil samples by Envirolab Services. Laboratory test certificates are provided in Attachment D.

2 General Site Details and Subsurface Conditions

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

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

Item	Comment
Topography	Within undulating terrain, mid slope of a northeast facing slope.
Typical Slopes, Aspect, Elevation	The site generally has a north easterly aspect with an overall grade <5 %. Site elevation ranges between approximately 52.5 mAHD in the northeast corner and 56.7 mAHD in the southwest corner of the site (Site Survey). Isolated areas across the north western portion of the site has easterly aspect, with grades up to approximately 10 %.
Expected Geology	Bringelly Shale comprising shale, carbonaceous claystone, laminite, fine to medium-grained lithic sandstone and rare coal (*Penrith 1:100 000 Geological Sheet 9030,* 1st edition)
Expected soil landscape	The NSW Office of Environment and Heritage's (OEH) information system (eSPADE) indicates the site to be located in the Blacktown (bt) soil landscape, consisting of gently undulating rises on Wianamatta Group shales. This soil landscape is characterised by > 200 cm of soil on lower side slopes. This soil landscape often associated with localised seasonal waterlogging, localised water erosion hazard, moderately reactive highly plastic subsoil, localised surface movement potential.
Existing Development	Currently undeveloped except a two storey brick house in the south eastern corner of the site. A single level timber board and a single level fibre board dwellings were formerly located in the central southern portion of the site, which have recently been demolished.
Vegetation	Grass, and scattered trees surrounding existing brick house. The northern portion of the site is more densely vegetated with large trees.
Neighbouring environment	The site is bordered by vacant land followed by Explorers Way to the south, residential properties to the east and west, and vacant land followed by M4 Motorway corridor to the north.
Drainage	Majority of the site drains via overland flow towards the northeast into a northeast-southwest aligned drainage depression extends near the northern portion of the site. The north western portion of the site drains via overland flow towards the east into the drainage depression.
Sub-surface Soil / Rock Units	Investigation revealed the following generalised subsurface units likely underlie site: <u>Unit A</u>: Topsoil comprising silty clay encountered generally in the northern portion of the site up to approximately 0.2 mbgl. <u>Unit B</u>: Fill comprising inferred poorly to moderately compacted silty clay / clayey silt, encountered in the southern and central portion of the site up to between approximately 0.2 mbgl (BH102) and 0.8 mbgl (BH103). It is expected that fill has been placed under uncontrolled conditions possibly as part of former development for landscaping and / or site levelling purposes. <u>Unit C</u>: Residual soil comprising: Unit C1: Firm to stiff silty clay encountered in the central and northern portions of the site up to approximately 1.55 mbgl. This unit was not encountered in the southern portion of the site. Unit C2: Very stiff to hard silty clay encountered below Unit C1 in the

Item	Comment
	central and northern portions and below Unit A in the southern portion of the site up to between approximately 3.2 mbgl and 4.2 mbgl (possibly extremely weathered rock from approximately 2.7 m, 2.9 m and 3.1 mbgl in BH105, BH106 and BH103, respectively). This unit contains interebedded extremely weathered rock and / or ironstone bands across the site. Unit D: Weathered and inferred very low to low strength shale / siltstone from depths of between 3.2 mbgl (BH104 and BH105) and 4.2 mbgl (BH101) up to maximum investigation termination depth of 6.6 mbgl.
Groundwater	Groundwater inflow was not encountered during drilling of the boreholes up to 6.6 mbgl. Ephemeral perched groundwater may be encountered within the soil profile originating from the seepage of ponding water in the drainage depression and / or infiltration of surface water during prolonged or intense rainfall events. Should further information on permanent site groundwater conditions be required, additional assessment would need to be carried out (i.e. installation of groundwater monitoring wells).

3 Geotechnical Assessment

3.1 Laboratory Testing

3.1.1 California Bearing Ratio (CBR) Testing

Laboratory CBR test results are summarised in Table 3 (refer Attachment D for CBR test certificate). CBR test results reported in GED, 2017 also reproduced in Table 3.

Table 3: CBR test results.

Borehole / Test Pit Number	Sample Depth (mbgl)	Material	CBR [1] Value (%)
BH101	0.3 – 0.8	Silty CLAY	2.0
BH107	0.3 – 0.8	Silty CLAY	5.0
TP1 [2]	0.3 – 1.1	Silty CLAY	1.5
TP2 [2]	0.2 – 0.6	Silty CLAY	4.0

Notes:

1. Four day soak, compacted to 98 % SMDD (±2 % of OMC), applying a 4.5 kg surcharge.
2. Reproduced from GED, 2017.

3.1.2 Salinity Classification

Laboratory test results for salinity classification are summarised in Table 4. Salinity classification test certificate is provided in Attachment D.

Table 4: Salinity test results.

Sample ID [1]	Material	$EC_{(1:5)}$ (dS/m)	EC_e (dS/m) [2]	Salinity Classification [3]
BH102/0.4-0.5	Silty CLAY	0.09	0.54	Non – Saline
BH102/1.4-1.5	Silty CLAY	0.25	1.50	Non – Saline
BH105/0.1-0.2	Silty CLAY	0.026	0.22	Non – Saline
BH105/0.7-0.8	Silty CLAY	0.21	1.26	Non – Saline
BH106/0.3-0.5	Silty CLAY	0.041	0.29	Non – Saline
BH106/1.0-1.1	Silty CLAY	0.14	0.98	Non – Saline

Notes:

1. Borehole#/Depth (mbgl).
2. Based on EC to EC_e multiplication factors from Table 6.1 in DLWC (2002).
3. Based on Table 6.2 of DLWC (2002) where EC_e <2 dS/m = non-saline, EC_e of 2-4 dS/m = slightly saline, EC_e of 4-8 dS/m = moderately saline, EC_e of 8-16 dS/m = very saline and EC_e of >16 S/m = highly saline.

3.1.3 Foundation Exposure Classification

Exposure classification results are summarised in Table 5 (refer Attachment D for laboratory test certificate). Exposure classification test results, as reported in GED, 2017, have also been reproduced in Table 5.

Table 5: Exposure classification test results.

Sample ID[1]	Material	EC_e (dS/m) [2]	pH	Sulphate (SO_4) (mg/kg)	Chloride (Cl) (mg/kg)	Exposure Classification		
						AS 2159 [3]	AS 2159 [4]	AS 3600 [5]
BH102/0.4-0.5	Silty CLAY	0.54	5.9	88	NA [7]	Non-aggressive	Mild	A1
BH102/1.4-1.5	Silty CLAY	1.50	5.2	140	NA [7]	Mild	Moderate	A2
BH105/0.1-0.2	Silty CLAY	0.22	5.8	< 10	NA [7]	Non-aggressive	Non-aggressive	A1
BH105/0.7-0.8	Silty CLAY	1.26	5.7	180	NA [7]	Non-aggressive	Moderate	A1
BH106/0.3-0.5	Silty CLAY	0.29	5.8	28	NA [7]	Non-aggressive	Non-aggressive	A1
BH106/1.0-1.1	Silty CLAY	0.98	5.7	180	NA [7]	Non-aggressive	Mild	A1
TP1/0.3-1.1 [6]	Silty CLAY	NA [7]	5.2	680	< 10	Mild	Mild	A2
TP2/0.6-1.0 [6]	Silty CLAY	NA [7]	5.1	630	220	Mild	Mild	A2

Notes:

1. Borehole or Test Pit #/Depth (mbgl).
2. From column 4 of Table 3.
3. Exposure classification for concrete piles in soil based on Table 6.4.2(C) of AS 2159 (2009).
4. Exposure classification for steel piles in soil based on Table 6.5.2(C) of AS 2159 (2009).
5. Exposure classification for buried reinforced concrete based on Tables 4.8.1 and 4.8.2 of AS 3600 (2018).
6. Reproduced from GED, 2017.
7. Test was not undertaken.

3.1.4 Atterberg Limits Testing (GED, 2017)

Atterberg limits test results presented in GED, 2017 are reproduced in Table 6.

Table 6: Summary of laboratory Atterberg Limits test results (GED, 2017).

Test Pit Number	Depth (mbgl)	Soil Type	Atterberg Limits (%)				Plasticity Classification	Potential Volume Change [2]
			LL[1]	PL[1]	PI[1]	LS[1]		
TP1	0.3-1.1	Silty CLAY	58	13	45	13.0	High	Medium to high
TP2	0.6-1.0	Silty CLAY	59	17	42	13.5	High	Medium to high
TP3	0.8	Silty CLAY	53	17	36	11.5	High	Medium to high

Notes:

1. LL = Liquid limit, PL= Plastic limit, PI=Plasticity index, LS = Linear shrinkage.
2. Based on Hazelton and Murphy, 2016.

Laboratory test results indicate that the tested residual soil samples are generally of high plasticity, which may result in moderate to high ground movement due to soil moisture changes.

3.2 Preliminary Material Properties

Preliminary material properties inferred from observations during borehole drilling, such as auger penetration resistance, DCP test results as well as engineering judgement are summarised in Table 7.

Table 7: Preliminary estimates of soil and rock strength properties.

Layer [1]	$\gamma_{in\text{-}situ}$ [2] (kN/m³)	Cu [3] (kPa)	C ' [4] (kPa)	Ø' [5] (deg)	E' [6] (MPa)
TOPSOIL / FILL [7]: Silty CLAY / Clayey SILT (moist)	18	NA [8]	NA [8]	NA [8]	NA [8]
RESIDUAL: Silty CLAY (firm to stiff, moist)	20	40	1	24	5
RESIDUAL: Silty CLAY (very stiff, moist)	20	100	4	26	15
RESIDUAL: Silty CLAY (hard and extremely weathered rock, moist)	21	200	6	26	25
WEATHERED ROCK: SHALE / SILTSTONE (inferred very low to low strength)	22	NA [8]	20	28	75

Notes:

1. Refer to borehole logs in Attachment B for material description details.
2. Material in-situ unit weight, based on visual assessment.
3. Average undrained shear strength estimate assuming normally consolidated clay.
4. Average drained cohesion.
5. Effective internal friction angle estimate, assuming drained conditions; may be dependent on rock defect conditions.
6. Average effective elastic modulus estimate, that should be adopted to calculate lateral deflection of pile under serviceability loading.
7. Inferred uncontrolled fill.
8. Not applicable.

3.3 Risk of Slope Instability

No evidence of former or current slope movement was observed at the site. We consider the risk to property and loss of life by potential slope instability, such as landslide or soil creep, to be very low subject to the recommendations in this report and adoption of relevant engineering standards and guidelines. A detailed slope risk assessment in accordance with Australian Geomechanics Society's Landslide Risk Management Guidelines (2007) was not undertaken.

4 Preliminary Pavement Thickness Design

4.1 Overview

Preliminary flexible pavement thicknesses design for the proposed car park was undertaken in accordance with Penrith City Council's Design Guidelines for Engineering Works for Subdivisions and Developments (PCC, 2013) and Australian Road Research Board, special report no. 41 (ARRB-SR41, 1989).

4.2 Design Parameters

4.2.1 Equivalent Standard Axles (ESA)

A traffic loading of 5×10^4 Equivalent Standard Axles (ESA) was adopted in accordance PCC, 2013.

4.2.2 Pavement Design Life

A design life of 20 years was adopted in the design in accordance PCC, 2013.

4.2.3 Design CBR

Test results returned CBR values of 1.5 % and 5.0 % for the residual subgrade soil. The variability in CBR values is likely due to variable soil consistency and variable gravel and silt content in the soil. Given the limited laboratory testing, a subgrade CBR of 1.5 % has been adopted to represent encountered site conditions. Therefore subgrade improvement / replacement will be required.

4.2.4 Subgrade Treatment

We recommend the following subgrade treatment options are adopted (following stripping of uncontrolled fill / topsoil to expose subgrade materials) to improve subgrade conditions for long term general use:

- Stabilise the initial subgrade layer (at least 300 mm thickness) with cement / lime or similar binding agent.

4.3 Pavement Thickness

Assuming adequate subgrade treatment, the pavement thickness design was carried out using ARRB-SR41 (1989) and adopting a CBR value of 3 %. Recommended pavement materials and material thicknesses are presented in Table 8.

Table 8: Pavement material thickness design for CBR 3 %.

Layer	Thickness (mm)	Total Thickness (mm)	Materials
Wearing Course	50	500	50 mm Asphalt Concrete (AC10) on a single coat hot bitumen flush seal
Base	175 [1]		DGB20
Sub-base	275 [1]		DGS40

Notes:

1. Based on Figure 7 of ARRB-SR41 (1989).

4.4 Earthworks

4.4.1 Subgrade Preparation

Prior to treatment, the subgrade is to be trimmed and compacted, following the removal of uncontrolled fill, topsoil and other unsuitable materials such as root containing soils. Minimum relative density of subgrade shall be 100 % Maximum Dry Density (MDD) at a standard compactive effort within -2 % and +2 % of optimum moisture content (OMC).

Prior to placement of pavement material, the treated subgrade shall be proof rolled and approved by a geotechnical engineer. If soft spots are encountered, they can be treated by one of the following methods subject to final design and adopted subgrade treatment option:

- Removal and replacement with approved fill under geotechnical engineer's direction.
- Further *in-situ* stabilisation with cement / lime or similar binding agent to a depth of at least 300 mm below finished level.

Use of stabilisation method and extent will depend on the condition of material to be stabilized.

4.4.2 Subsoil Drainage

Surface and sub-soil drainage is to be provided in accordance with Council requirements. Typically, subsurface drains are installed on the upslope side of all internal roads or on both sides where adjacent to vegetated areas, and generally extend 600 mm below pavement level. Austroads advises against extending subsurface drainage into highly reactive soils beneath the pavement.

4.4.3 Placement and Testing of Pavement Material

Pavement materials shall be placed in layers (when compacted) not thicker than 250 mm or less than 75 mm. Pavement materials shall be compacted to the following condition:

- Sub-base - Minimum 95 % 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 Penrith City Council Engineering Construction Specification for Civil Works (PCC, 2017). Testing to be carried out at the rate of one test per 50 lineal metres of road, with a minimum of two tests on any one road. 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.

4.4.4 Fill Placement

Should filling be required to raise subgrade levels, suitable well graded granular material approved for use by a Geotechnical Engineer is recommended for structural fill. Site-won excavated residual soils are not recommended for re-use as structural fill due to their moderate to high reactivity to soil moisture variation and associated difficulties in placement. Proof rolling of subgrade should be closely monitored by the project geotechnical engineer to detect soft or unstable areas which should be removed and replaced with engineered fill or alternatively stabilised or bridged.

All earthwork and fill material testing and preparation is to be approved by a Geotechnical Engineer and undertaken in accordance with AS 3798 (2007) and Penrith City Council's *Design Guidelines for Engineering Works for Subdivisions and Developments (2013)*.

4.5 Other Considerations

Transitioning of pavement material between the existing and new pavement sections needs to be included in detailed design, ensuring adequate offset between wheel paths and transition zones.

5 Geotechnical Recommendations

5.1 General Geotechnical Recommendations

General geotechnical recommendations are provided in Attachment E. Additional preliminary geotechnical recommendations for the proposed development are provided below.

5.2 Excavation Support

The proposed development does not involve bulk excavation. Minor cutting for site levelling and excavations for foundations exceeding 1.0 m depth, if required, may be temporarily (unsupported for less than 1 month) battered back with a maximum batter grade of 1V:1.5H. Permanent batters, if adopted, should not exceed a grade of 1V:3H. Recommended batters are subject to inspection and approval by an experienced geotechnical engineer on site.

5.3 Footings and Foundations

Variable foundation material will likely be exposed in the southern (e.g. very stiff to hard residual soil) and central and eastern portion (e.g. firm to stiff residual soil) of the site following stripping off topsoil / uncontrolled fill. Shallow footings, such as pad and strip footings founding on at least very stiff residual soil at a minimum depth of 0.8 mbgl may be adopted as support for new structures at the southern area of the site. Deepening (e.g. in the central and eastern portion) of foundations (e.g. screw piles / piers) to at least 1.8 m will be required to obtain desired very stiff consistency into the residual soil. Subject to founding in at least very stiff residual soil, an allowable bearing capacity (ABC) of 150 kPa may be adopted for the design of shallow footings and 300kPa for piles embedded at least 3 pile diameters into material unit. We recommend all footings within building are founded within consistent material to minimise risk of differential foundation movement.

Higher bearing capacity may be achieved by adopting piles founding into hard residual clay or shale bedrock. An allowable bearing capacities of 400 kPa and 700 kPa may be adopted for piles embedded at least 3 pile diameters into hard clay and 0.5 m into very low to low strength rock, respectively.

Piles may socketed into higher strength rock to accommodate higher end bearing pressures, if required. Further investigation would need to be carried out, such as rock coring and laboratory point load testing, to further assess rock condition at the site.

Footings should be designed by a suitably qualified and experienced structural or geotechnical engineer. Consideration should be given to the potential ground surface movement associated with highly reactive soil during design of at grade structures (e.g. pavement, footpath) and shallow footings.

5.4 Foundations Movement

The residual silty clay underlying the site is typically subject to softening and high shrink-swell due to soil moisture changes. This is more likely across the areas in the vicinity of the drainage channel. Subject to foundation design as recommended in Section 5.3 and provision of appropriate surface and sub-surface drainage to divert overland flows and potential perched groundwater, away from excavations, retaining walls or foundations, both total and differential movements of structures should be within tolerable limits.

5.5 Earth Pressure Coefficients

Retaining wall design, if required, may adopt preliminary active, at rest and passive earth pressure coefficients, respectively, of:

1. 0.59, 0.42, 2.37 for existing fill and firm to stiff residual soil.
2. 0.56, 0.39, 2.56 for very stiff residual sand.
3. 0.50, 0.33, 3.00 for hard residual soil and extremely weathered rock.

5.6 Site Classifications

The site is classified as a class 'P' site in accordance with AS 2870 (2011) due to presence of uncontrolled fill and unsuitable (e.g. firm silty clay) foundation material at the foundation level across the site. A reclassification to "H1" may be possible for lightly loaded shallow footings founding on at least very stiff residual soil.

These site classifications are subject to the recommendations presented in this report, the design of footings in accordance with the relevant Australian Standards and industry guidelines.

5.7 Exposure Classification

Based on laboratory test results provided in Table 5 and in accordance with AS2159 (2009), an exposure classification of 'Mild' and 'Moderate' should be adopted for buried concrete and steel piles, respectively. In accordance with AS3600 (2018), an exposure classification of 'A2' should be adopted for shallow concrete footings founding on residual soil.

5.8 Soil Salinity

Sub-surface materials at the site can generally be categorised as non-saline. No specific saline soil management strategies are likely to be required. However, further testing may need to be undertaken (particularly at depth), depending on the final development levels, to delineate extent of saline soils, particularly in the northern portion of the site.

5.9 Earthworks

Should re-use of excavated residual soils be considered for site filling, we recommend limiting this material re-use to general fill / landscape areas, given the high plasticity of the residual soils. We recommend the use of approved well graded granular material as structural fill.

Fill placement should be undertaken in layers not more than 300 mm in loose thickness following removal of topsoil and other unsuitable materials such as root containing soils or uncontrolled fill and subsequent proof rolling. A qualified geotechnical engineer should inspect and approve earthworks.

All earthworks should be carried out in accordance with AS3798 (2007). A qualified geotechnical engineer should inspect and approve earthworks.

5.10 Drainage requirements

Appropriate surface and sub-surface drainage should be provided to divert overland flows and potential perched groundwater, away from excavations, retaining walls or foundations and limit ponding of water, particularly in excavations or near footings. Expected limited seepage inflow into excavations can likely be managed by sump and pump provision during construction. Collected water should be discharged into council approved stormwater systems downslope of the site.

5.11 Further Works

The following additional assessments should be undertaken to progress design, documentation and construction of the proposed aged care facility:

1. Assessment of specific foundation conditions, including assessment of rock depths and conditions, below foundation levels, if higher end bearing capacity is required. This should include rock coring and point load testing of collected rock samples to assess rock strength.

2. Adopted CBR values should be confirmed once details of proposed developments are known (e.g. the proposed cut / fill and final development levels).

3. Additional geotechnical investigations, if required, for other areas not accessible to date.

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

5. Inspection of foundation by an experienced geotechnical engineer during construction to confirm design bearing capacity is achieved at foundation level.

6 References

Australian Road Research Board (1989) *A Structural Design Guide for Flexible Residential Street Pavements*, Special report no. 41 (ARRB-SR41, 1989).

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Penrith City Council (2013) *Design Guidelines for Engineering Works for Subdivisions and Developments* (PCC, 2013).

Penrith City Council (2017) *Engineering Construction Specification for Civil Works* (PCC, 2017).

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

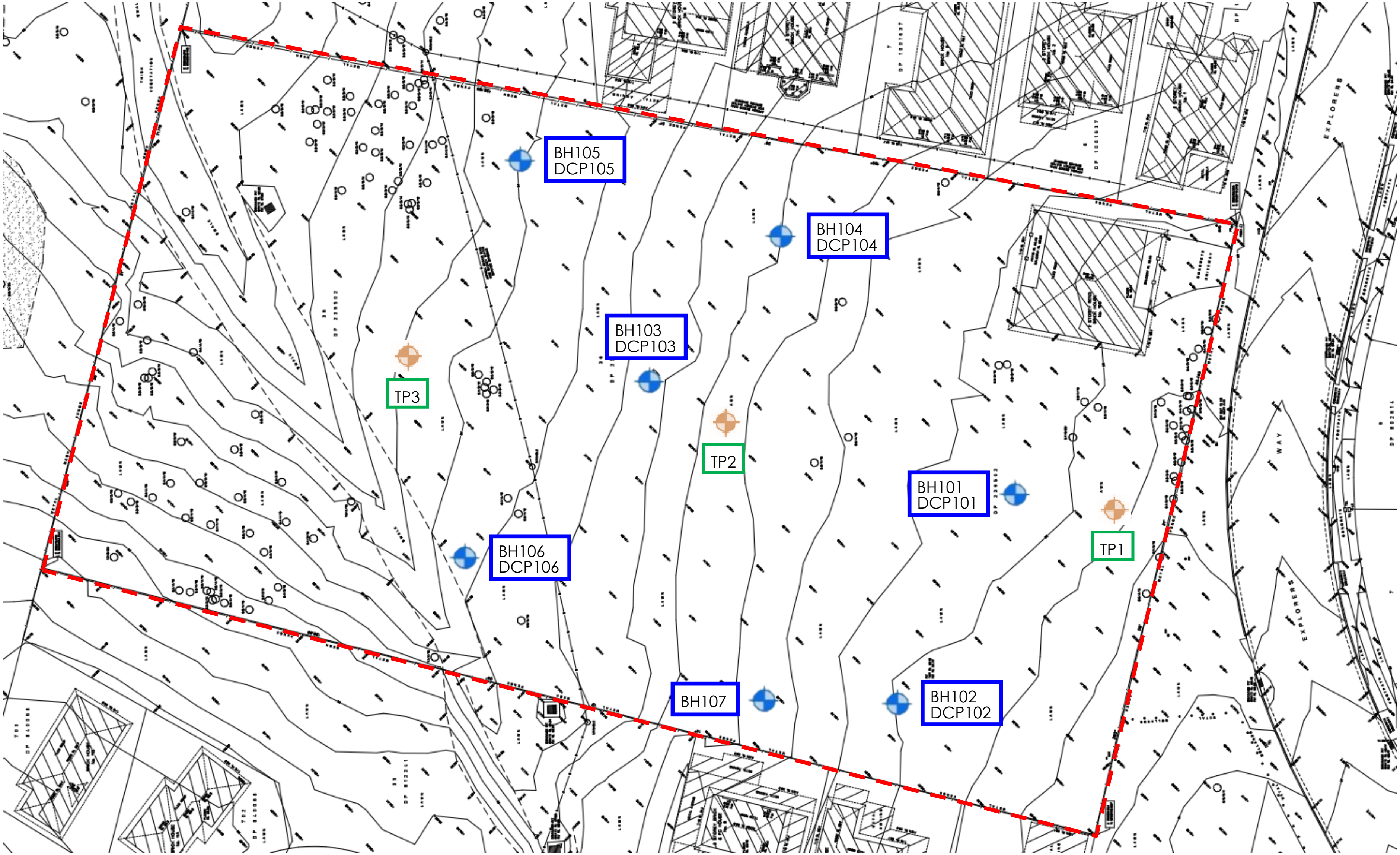
Standards Australia Limited (2009) AS 2159:2009, *Piling – Design and installation*, SAI Global Limited.

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

Standards Australia Limited (2009) AS 3600:2018, *Concrete Structures*, 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



Key:

Indicative site boundary

Approximate borehole and DCP test location (by MA)

Indicative test pit location (GED, 2017)

martens

Martens & Associates Pty Ltd ABN 85 070 240 890	**Environment | Water | Wastewater | Geotechnical | Civil | Management**
Drawn:	WB
Approved:	KB
Date:	09.03.2021
Scale:	NA

SITE LAYOUT AND GEOTECHNICAL TESTING PLAN
94-100 Explorers Way, St Clair, NSW
(Source: Provided by client)

Drawing No:
FIGURE 1

Project No: P2007910JR02V01

8 Attachment B – Test Borehole Logs

CLIENT	Opal Aged Care c/- Pact PM	COMMENCED	06/10/2020	COMPLETED	06/10/2020	**REF BH101**
PROJECT	Geotechnical Assessment	LOGGED	WB	CHECKED	KB	Sheet 1 OF 1
SITE	94-100 Explorers Way, St Clair, NSW	GEOLOGY	Bringelly Shale	VEGETATION	Grass	PROJECT NO. P2007910
EQUIPMENT	4WD ute-mounted hydraulic drill rig	EASTING	150.801017	RL SURFACE	55.9 m	DATUM: AHD
EXCAVATION DIMENSIONS	Ø100 mm x 6.60 m depth	NORTHING	-33.791952	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-M	Not Encountered	0	55.90	0.1-0.3/S/1 D 0.10-0.30 m			CI-CH	FILL: Silty CLAY; medium to high plasticity; dark brown, red; trace road base gravels; trace concrete and tile fragments; inferred poorly to moderately compacted.	M (<<PL)		FILL
				0.30 / 55.60	0.3-0.8/CBR/1 D 0.30-0.80 m; 0.6-0.8/S/1 D 0.60-0.80 m			CH	Silty CLAY; high plasticity; red and brown; trace fine gravels.		VSt	RESIDUAL SOIL
	H		1	1.00 / 54.90	1.1-1.2/S/1 D 1.10-1.20 m; 1.2-1.3/R/1 D 1.20-1.30 m				Brown, yellow-brown, pale grey; ironstone band (approximately 200 mm thick).		H	1.20: V-bit refusal.
AD/T	L			1.80 / 54.10	1.8-1.9/R/1 D 1.80-1.90 m				Red-brown, grey.			
			2	2.20 / 53.70					Grey.			
	L-M		4	4.20 / 51.70	4.2-4.4/R/1 D 4.20-4.40 m				SHALE/SILTSTONE; grey, pale grey, brown; highly weathered; inferred very low strength.			WEATHERED ROCK
	M		5	4.80 / 51.10	4.8-4.9/R/1 D 4.80-4.90 m				Brown.			
	H		6	6.20 / 49.70					Grey and brown, inferred very low to low strength.			
			7	6.60					Hole Terminated at 6.60 m			6.60: TC-bit refusal on inferred low to medium strength shale/siltstone.

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

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Engineering Log - BOREHOLE

CLIENT	Opal Aged Care c/- Pact PM	COMMENCED	06/10/2020	COMPLETED	06/10/2020	REF BH102
PROJECT	Geotechnical Assessment	LOGGED	WB	CHECKED	KB	Sheet 1 OF 1
SITE	94-100 Explorers Way, St Clair, NSW	GEOLOGY	Bringelly Shale	VEGETATION	Grass	PROJECT NO. P2007910
EQUIPMENT	4WD ute-mounted hydraulic drill rig	EASTING	150.800828	RL SURFACE	56 m	DATUM AHD
EXCAVATION DIMENSIONS	Ø100 mm x 5.50 m depth	NORTHING	-33.791834	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		56.00				ML	FILL: Clayey SILT; low plasticity; dark brown; trace fine to medium road base and ironstone gravels; trace roots; inferred poorly to moderately compacted.			FILL
				0.20 / 55.80	0.1-0.2/S/1 D 0.10-0.20 m			CH	Silty CLAY; high plasticity; brown, red-brown, red; trace fine ironstone gravels.	M (<<PL)		RESIDUAL SOIL
					0.4-0.5/S/1 D 0.40-0.50 m						VSt - H	
				0.80 / 55.20					Red, red-brown, grey.			
	H		1	1.20 / 54.80	1.1-1.2/S/1 D 1.10-1.20 m				Brown, red-brown.			
				1.50 / 54.50	1.4-1.5/S/1 D 1.40-1.50 m				Brown, red-brown, pale grey (possibly ironstone band).			1.50: V-bit refusal.
AD/T	L											
	H		2	2.00 / 54.00	2.0-2.2/R/1 D 2.00-2.20 m				2.0m-2.2m: Ironstone band.			
	L		3							M (<PL)	H	
	M		4	4.00 / 52.00	4.0-4.2/R/1 D 4.00-4.20 m				SHALE/SILTSTONE; pale grey, grey, red-brown; highly weathered; inferred very low strength.			WEATHERED ROCK
	H		5	5.00 / 51.00					Inferred very low to low strength.			
				5.50					Hole Terminated at 5.50 m			5.50: TC-bit refusal on inferred low to medium strength shale/siltstone.
			6									
			7									

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

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Engineering Log - BOREHOLE

CLIENT	Opal Aged Care c/- Pact PM	COMMENCED	06/10/2020	COMPLETED	06/10/2020	**REF BH103**	
PROJECT	Geotechnical Assessment	LOGGED	WB	CHECKED	KB	Sheet 1 OF 1	
SITE	94-100 Explorers Way, St Clair, NSW	GEOLOGY	Bringelly Shale	VEGETATION	Grass	PROJECT NO. P2007910	
EQUIPMENT	4WD ute-mounted hydraulic drill rig	EASTING	150.801144	RL SURFACE	54.7 m	DATUM	AHD
EXCAVATION DIMENSIONS	Ø100 mm x 5.80 m depth	NORTHING	-33.791537	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	M	Not Encountered	0.00	54.70	0.2-0.3/S/1 D 0.20-0.30 m; 0.7/S/1 D 0.70 m	ML	FILL: Clayey SILT; low plasticity; dark brown; trace fine road base and ironstone gravels; trace roots; inferred poorly to moderately compacted.	M (<<PL)		FILL
AD/V	L	Not Encountered	0.80	53.90	0.9-1.0/S/1 D 0.90-1.00 m; 1.5/S/1 D 1.50 m	CH	Silty CLAY; high plasticity; brown, red-brown, red; trace fine ironstone gravels.		F; St; VSt	RESIDUAL SOIL
AD/V	L	Not Encountered	2.00	52.70	2.0/S/1 D 2.00 m	CH	Brown, yellow-brown, pale grey.	M (<PL)	H	
AD/V	M	Not Encountered	2.50	52.20	2.5/S/1 D 2.50 m	CH	Brown and pale grey.		H	
AD/V	H	Not Encountered	3.10	51.60		CH	Possibly extremely weathered rock.		H	
AD/T	M	Not Encountered	3.50	51.20	4.5/R/1 D 4.50 m		SHALE/SILTSTONE; brown, red-brown, grey, pale grey; highly weathered; inferred very low strength.			WEATHERED ROCK 3.50: V-bit refusal.
AD/T	H	Not Encountered	4.70	50.00			Brown; inferred very low to low strength.			
			5.80				Hole Terminated at 5.80 m			5.80: TC-bit refusal on inferred low to medium strength.

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

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Engineering Log - BOREHOLE

CLIENT	Opal Aged Care c/- Pact PM	COMMENCED	06/10/2020	COMPLETED	06/10/2020	**REF BH104**	
PROJECT	Geotechnical Assessment	LOGGED	WB	CHECKED	KB	Sheet 1 OF 1	
SITE	94-100 Explorers Way, St Clair, NSW	GEOLOGY	Bringelly Shale	VEGETATION	Grass	PROJECT NO. P2007910	
EQUIPMENT	4WD ute-mounted hydraulic drill rig	EASTING	150.801524	RL SURFACE	55.2 m	DATUM	AHD
EXCAVATION DIMENSIONS	Ø100 mm x 5.20 m depth	NORTHING	-33.791739	ASPECT	North	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		55.20	0.0-0.1/S/1 0.00-0.10 m	CI	TOPSOIL: Silty CLAY; medium plasticity; brown, red-brown; trace roots; trace fine ironstone and shale gravels.			TOPSOIL
				0.20 / 55.00	0.6-0.7/S/1 D 0.60-0.70 m	CI-CH	Silty CLAY; medium to high plasticity; red-brown, red; trace ironstone gravels.	M (<<PL)	VSt / St	RESIDUAL SOIL
			1	1.20 / 54.00	1.4-1.5/S/1 D 1.40-1.50 m		Red.	M (<PL)	VSt	
	M			1.70 / 53.50	1.8-1.9/S/1 D 1.80-1.90 m		Brown and dark grey; inferred hard.		H	
	M-H									
AD/T	H		2	2.00 / 53.20			Ironstone band.			2.00: V-bit refusal on inferred ironstone band.
	L			2.20 / 53.00	2.4/S/1 D 2.40 m	CH	Silty CLAY; high plasticity; brown, red-brown, pale grey; trace fine ironstone gravels.	M (<PL)	H	
	H		3	3.20 / 52.00			SHALE/SILTSTONE; brown, dark grey; highly weathered; very low to low strength.			WEATHERED ROCK
	M		4							
	H			4.50 / 50.70			Inferred very low to low strength.			
			5	5.20	5.1-5.2/R/1 D 5.10-5.20 m		Hole Terminated at 5.20 m			5.20: TC-bit refusal on inferred low to medium shale/siltstone.
			6							
			7							

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Engineering Log - BOREHOLE

MARTENS 2.00 LIB.GLB Log MARTENS BOREHOLE P2007910BH101-BH107V01.GPJ <<DrawingFile>> 04/11/2020 10:24 8.30.004 Datgel Lab and In Situ Tool - DGD | Lib: Martens 2.00 2016-11-13 Prj: Martens 2.00 2016-11-13

CLIENT	Opal Aged Care c/- Pact PM	COMMENCED	06/10/2020	COMPLETED	06/10/2020	**REF BH105**	
PROJECT	Geotechnical Assessment	LOGGED	WB	CHECKED	KB	Sheet 1 OF 1	
SITE	94-100 Explorers Way, St Clair, NSW	GEOLOGY	Bringelly Shale	VEGETATION	Grass	PROJECT NO. P2007910	
EQUIPMENT	4WD ute-mounted hydraulic drill rig	EASTING	150.801633	RL SURFACE	54 m	DATUM	AHD
EXCAVATION DIMENSIONS	Ø100 mm x 5.70 m depth	NORTHING	-33.791391	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	Not Encountered	0.00	54.00	0.1-0.2/S/1 D 0.10-0.20 m	CL	TOPSOIL: Silty CLAY; low plasticity; dark brown; trace roots; trace fine ironstone gravels.		St	TOPSOIL
	L		0.20	53.80	0.4-0.5/S/1 D 0.40-0.50 m	CL-CI	Silty CLAY; low to medium plasticity; dark brown, red-brown; trace fine ironstone gravels.	M (<PL)	VSt	RESIDUAL SOIL
	L		0.70	53.30	0.7-0.8/S/1 D 0.70-0.80 m	CH	Silty CLAY; high plasticity; brown, red-brown, red; trace fine ironstone gravels.		F	
	L / M		1.30	52.70	1.3/S/1 D 1.30 m		Red, grey.		St - VSt	
	M		2.00	52.00	2.0/S/1 D 2.00 m		Brown, orange-brown.		VSt	
	M		2.40	51.60			Brown, yellow-brown, grey, pale grey.			
	H		2.70	51.30	3.1-3.2/S/1 D 3.10-3.20 m		Possibly extremely weathered rock.		H	
AD/T	M / H / M		3.20	50.80	3.7-3.9/R/1 D 3.70-3.90 m		SHALE/SILTSTONE; brown and grey; highly weathered; very low strength.			WEATHERED ROCK 3.20: V-bit refusal.
	H		5.20	48.80	5.5/R/1 D 5.50 m		Inferred very low to low strength.			
			5.70				Hole Terminated at 5.70 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	Opal Aged Care c/- Pact PM	COMMENCED	06/10/2020	COMPLETED	06/10/2020	**REF BH106**	
PROJECT	Geotechnical Assessment	LOGGED	WB	CHECKED	KB	Sheet 1 OF 1	
SITE	94-100 Explorers Way, St Clair, NSW	GEOLOGY	Bringelly Shale	VEGETATION	Grass	PROJECT NO. P2007910	
EQUIPMENT	4WD ute-mounted hydraulic drill rig	EASTING	150.801048	RL SURFACE	54.2 m	DATUM	AHD
EXCAVATION DIMENSIONS	Ø100 mm x 5.30 m depth	NORTHING	-33.791355	ASPECT	Northwest	SLOPE	<5%

Drilling				Sampling			Field Material Description				
METHOD	PENETRATION RESISTANCE	WATER	DEPTH (metres)	DEPTH RL	SAMPLE OR FIELD TEST	RECOVERED	USCS / ASCS CLASSIFICATION	SOIL/ROCK MATERIAL DESCRIPTION	MOISTURE CONDITION	CONSISTENCY DENSITY	STRUCTURE AND ADDITIONAL OBSERVATIONS
AD/V	M	Not Encountered		54.20 / 0.20			CI	TOPSOIL: Silty CLAY; medium plasticity; dark brown, red-brown; trace roots; trace fine ironstone gravels.			TOPSOIL
				54.00	0.3-0.5/S/1 D 0.30-0.50 m	Yes	CI	Silty CLAY; medium plasticity; dark brown, red-brown; trace fine ironstone gravels.	M (<<PL)	VSt and H	RESIDUAL SOIL
			1		1.0-1.1/S/1 D 1.00-1.10 m	Yes				St	
				1.20 / 53.00				Brown, red-brown.			
	H			1.50 / 1.60	1.5-1.6/S/1 D 1.50-1.60 m	Yes		Ironstone band.			
AD/T	L			52.60			CH	Silty CLAY; high plasticity; brown, red-brown, orange, red.	M (<PL)	VSt and H	1.60: V-bit refusal on inferred ironstone band.
			2	2.20 / 52.00				Brown, red-brown, pale grey.			
				2.40 / 51.80	2.5/S/1 D 2.50 m			Pale grey.			
			3	2.90 / 51.30				Brown and grey (possibly extremely weathered rock).		H	
	M			3.40 / 50.80	3.7/R/1 D 3.70 m			SHALE/SILTSTONE; brown, yellow-brown, grey; highly weathered; inferred very low strength.			WEATHERED ROCK
	H		4	4.50 / 49.70	4.7-5.0/R/1 D 4.70-5.00 m	Yes		Inferred very low to low strength.			
			5	5.10 / 49.10				Grey.			
				5.30				Hole Terminated at 5.30 m (Target depth reached)			
			6								
			7								

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

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Engineering Log - BOREHOLE

CLIENT	Opal Aged Care c/- Pact PM	COMMENCED	06/10/2020	COMPLETED	06/10/2020	REF BH107	
PROJECT	Geotechnical Assessment	LOGGED	WB	CHECKED	KB	Sheet 1 OF 1	
SITE	94-100 Explorers Way, St Clair, NSW	GEOLOGY	Bringelly Shale	VEGETATION	Grass	PROJECT NO. P2007910	
EQUIPMENT	4WD ute-mounted hydraulic drill rig	EASTING	150.80084	RL SURFACE	55.3 m	DATUM	AHD
EXCAVATION DIMENSIONS	Ø100 mm x 1.00 m depth	NORTHING	-33.791666	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		55.30				ML	FILL: Clayey SILT; low plasticity; dark brown; trace fine to medium road base and ironstone gravels; trace roots; inferred poorly to moderately compacted.	M (<<PL)		FILL
				0.30 55.00	0.3-0.8/CBR/1 CBR 0.30-0.80 m			CH	Silty CLAY; high plasticity; brown, red-brown, red; trace fine ironstone gravels; inferred very stiff.	M (<PL)	VSt	RESIDUAL SOIL
			1	1.00					Hole Terminated at 1.00 m (Target depth reached)			
			2									
			3									
			4									
			5									
			6									
			7									

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Engineering Log - BOREHOLE

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9 Attachment C – DCP ‘N’ Counts

Dynamic Cone Penetrometer Test Log Summary

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Site	Proposed Opal Aged Care Centre – 94-100 Explorers Way, St Clair, NSW	**DCP Group Reference**	P2007910JS01V01
Client	Opal Aged Care c/ - Pact PM	**Log Date**	06.10.2020
Logged by	WB	**Page**	1 of 1
Checked by	SK		
Comments	DCP commenced from 50 mm bgl.		

TEST DATA

Depth Interval (m)	DCP101	DCP102	DCP103	DCP104	DCP105	DCP106		
0.15	10	18	6	10	4	12		
0.30	25	16	29	12	5	25		
0.45	10	16	16	13	9	18		
0.60	9	16	19	11	9	16		
0.75	11	16	14	6	12	10		
0.90	13	14	12	5	3	5		
1.05	20	14	3	5	3	13		
1.20	20	17	3	5	3	19		
1.35	18	31	4	5	7	10		
1.50	29	33	6	8	8	20		
1.65	22	Terminated @ 1.55 m due to high 'N' counts	10	23	10	23		
1.80	Terminated @ 1.7 m due to high 'N' counts		10	36 / 100 mm	11	7		
1.95			15	Terminated due to bounce @ 1.8 m	11	11		
2.10			17		11	10		
2.25			25		17	12		
2.40			39		11	15		
2.55			Terminated @ 2.45 m due to high 'N' counts		12	14		
2.70					13	23		
2.85					28	18		
3.00					27	26		
3.15					Terminated @ 3.05 m due to high 'N' counts	36		
3.30						39		
3.45						Terminated due to bounce @ 3.35 m		

10 Attachment D – Laboratory Test Certificates

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: P2007910
Location: 94-100 Explorers Way, St Clair NSW

Job number: 20-0079
Report number: 1
Page: 1 of 1

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.	22729			
Customer sample no.	7910/BH101/ CBR01/0.3-0.8			
Date sampled	06/10/2020			
Material description	silty CLAY, trace of gravel, red/grey			
Maximum dry density (t/m^3)	1.71			
Optimum moisture content (%)	21.0			
Field moisture content (%)	n/a			
Oversize retained on 19.0mm sieve (%)	2			
Minimum curing time (hours)	96			
Dry density before soak (t/m^3)	1.67			
Dry density after soak (t/m^3)	1.63			
Moisture content before soak (%)	20.7			
Moisture content after soak (%)	22.7			
Moisture content after test - top 30mm (%)	29.1			
Moisture content after test - remaining depth (%)	21.4			
Density ratio before soaking (%)	98.0			
Moisture ratio before soaking (%)	98.5			
Period of soaking (days)	4			
Compactive effort	Standard			
Mass of surcharge applied (kg)	4.5			
Swell after soaking (%)	2.5			
Penetration (mm)	2.5			
CBR Value (%)	**2.0**			

Notes: Specified LDR: 98 ±1%

Method of establishing plasticity level - Visual / tactile

Approved Signatory: C. Greely **Date:** 27/10/2020

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Accredited for compliance with ISO/IEC 17025 - Testing.

NATA Accredited Laboratory Number: **17062**

resource laboratories
AGGREGATE, ROCK, AND SOIL TESTING
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Sydney: 12/1 Boden Road Seven Hills NSW 2147 | PO Box 45 Pendle Hill NSW 2145
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Test Report

Customer: Martens & Associates Pty Ltd **Job number:** 20-0079
Project: P2007910 **Report number:** 2
Location: 94-100 Explorers Way, St Clair, NSW **Page:** 1 of 1

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.	22730			
Customer sample no.	7910/BH107/ CBR02/0.3-0.8			
Date sampled	06/10/2020			
Material description	silty CLAY, trace of gravel, brown/ red/yellow-brown			
Maximum dry density (t/m^3)	1.73			
Optimum moisture content (%)	22.1			
Field moisture content (%)	n/a			
Oversize retained on 19.0mm sieve (%)	0			
Minimum curing time (hours)	168			
Dry density before soak (t/m^3)	1.69			
Dry density after soak (t/m^3)	1.68			
Moisture content before soak (%)	21.9			
Moisture content after soak (%)	23.6			
Moisture content after test - top 30mm (%)	27.1			
Moisture content after test - remaining depth (%)	23.1			
Density ratio before soaking (%)	98.0			
Moisture ratio before soaking (%)	99.0			
Period of soaking (days)	4			
Compactive effort	Standard			
Mass of surcharge applied (kg)	4.5			
Swell after soaking (%)	1.0			
Penetration (mm)	2.5			
CBR Value (%)	**5.0**			

Notes: Specified LDR: 98 ±1%

Method of establishing plasticity level - Visual / tactile

Approved Signatory: C. Greely **Date:** 03/11/2020

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NATA Accredited Laboratory Number: **17062**

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CERTIFICATE OF ANALYSIS 252910

Client Details

Client	Martens & Associates Pty Ltd
Attention	Waisul Bari, Jeff Fulton
Address	Suite 201, 20 George St, Hornsby, NSW, 2077

Sample Details

Your Reference	**P2007910, Geotechnical Assess: 100 Explorers Way**
Number of Samples	6 Soil
Date samples received	08/10/2020
Date completed instructions received	08/10/2020

Analysis Details

Please refer to the following pages for results, methodology summary and quality control data.

Samples were analysed as received from the client. Results relate specifically to the samples as received.

Results are reported on a dry weight basis for solids and on an as received basis for other matrices.

Report Details

Date results requested by	15/10/2020
Date of Issue	14/10/2020

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Accredited for compliance with ISO/IEC 17025 - Testing. **Tests not covered by NATA are denoted with ***

Results Approved By
Diego Bigolin, Team Leader, Inorganics

Authorised By

Nancy Zhang, Laboratory Manager

ACCREDITED FOR TECHNICAL COMPETENCE

Misc Inorg - Soil						
Our Reference		252910-1	252910-2	252910-3	252910-4	252910-5
Your Reference	UNITS	7910/BH102	7910/BH102	7910/BH105	7910/BH105	7910/BH106
Depth		0.4-0.5	1.4-1.5	0.1-0.2	0.7-0.8	0.3-0.5
Date Sampled		06/10/2020	06/10/2020	06/10/2020	06/10/2020	06/10/2020
Type of sample		Soil	Soil	Soil	Soil	Soil
Date prepared	-	12/10/2020	12/10/2020	12/10/2020	12/10/2020	12/10/2020
Date analysed	-	12/10/2020	12/10/2020	12/10/2020	12/10/2020	12/10/2020
pH 1:5 soil:water	pH Units	5.9	5.2	5.8	5.7	5.8
Electrical Conductivity 1:5 soil:water	µS/cm	90	250	26	210	41
Sulphate, SO4 1:5 soil:water	mg/kg	88	140	<10	180	28

Misc Inorg - Soil		
Our Reference		252910-6
Your Reference	UNITS	7910/BH106
Depth		1.0-1.1
Date Sampled		06/10/2020
Type of sample		Soil
Date prepared	-	12/10/2020
Date analysed	-	12/10/2020
pH 1:5 soil:water	pH Units	5.7
Electrical Conductivity 1:5 soil:water	µS/cm	140
Sulphate, SO4 1:5 soil:water	mg/kg	180

Method ID	Methodology Summary
Inorg-001	pH - Measured using pH meter and electrode in accordance with APHA latest edition, 4500-H+. Please note that the results for water analyses are indicative only, as analysis outside of the APHA storage times.
Inorg-002	Conductivity and Salinity - measured using a conductivity cell at 25°C in accordance with APHA latest edition 2510 and Rayment & Lyons.
Inorg-081	Anions - a range of Anions are determined by Ion Chromatography, in accordance with APHA latest edition, 4110-B. Waters samples are filtered on receipt prior to analysis. Alternatively determined by colourimetry/turbidity using Discrete Analyser.

QUALITY CONTROL: Misc Inorg - Soil					Duplicate				Spike Recovery %	
Test Description	Units	PQL	Method	Blank	#	Base	Dup.	RPD	LCS-1	[NT]
Date prepared	-			12/10/2020	2	12/10/2020	12/10/2020		12/10/2020	[NT]
Date analysed	-			12/10/2020	2	12/10/2020	12/10/2020		12/10/2020	[NT]
pH 1:5 soil:water	pH Units		Inorg-001	[NT]	2	5.2	5.2	0	101	[NT]
Electrical Conductivity 1:5 soil:water	µS/cm	1	Inorg-002	<1	2	250	260	4	105	[NT]
Sulphate, SO4 1:5 soil:water	mg/kg	10	Inorg-081	<10	2	140	130	7	105	[NT]

Result Definitions

NT	Not tested
NA	Test not required
INS	Insufficient sample for this test
PQL	Practical Quantitation Limit
<	Less than
>	Greater than
RPD	Relative Percent Difference
LCS	Laboratory Control Sample
NS	Not specified
NEPM	National Environmental Protection Measure
NR	Not Reported

Quality Control Definitions

Blank	This is the component of the analytical signal which is not derived from the sample but from reagents, glassware etc, can be determined by processing solvents and reagents in exactly the same manner as for samples.
Duplicate	This is the complete duplicate analysis of a sample from the process batch. If possible, the sample selected should be one where the analyte concentration is easily measurable.
Matrix Spike	A portion of the sample is spiked with a known concentration of target analyte. The purpose of the matrix spike is to monitor the performance of the analytical method used and to determine whether matrix interferences exist.
LCS (Laboratory Control Sample)	This comprises either a standard reference material or a control matrix (such as a blank sand or water) fortified with analytes representative of the analyte class. It is simply a check sample.
Surrogate Spike	Surrogates are known additions to each sample, blank, matrix spike and LCS in a batch, of compounds which are similar to the analyte of interest, however are not expected to be found in real samples.

Australian Drinking Water Guidelines recommend that Thermotolerant Coliform, Faecal Enterococci, & E.Coli levels are less than 1cfu/100mL. The recommended maximums are taken from "Australian Drinking Water Guidelines", published by NHMRC & ARMC 2011.

The recommended maximums for analytes in urine are taken from “2018 TLVs and BEIs”, as published by ACGIH (where available). Limit provided for Nickel is a precautionary guideline as per Position Paper prepared by AIOH Exposure Standards Committee, 2016.

Guideline limits for Rinse Water Quality reported as per analytical requirements and specifications of AS 4187, Amdt 2 2019, Table 7.2

Laboratory Acceptance Criteria

Duplicate sample and matrix spike recoveries may not be reported on smaller jobs, however, were analysed at a frequency to meet or exceed NEPM requirements. All samples are tested in batches of 20. The duplicate sample RPD and matrix spike recoveries for the batch were within the laboratory acceptance criteria.

Filters, swabs, wipes, tubes and badges will not have duplicate data as the whole sample is generally extracted during sample extraction.

Spikes for Physical and Aggregate Tests are not applicable.

For VOCs in water samples, three vials are required for duplicate or spike analysis.

Duplicates: >10xPQL - RPD acceptance criteria will vary depending on the analytes and the analytical techniques but is typically in the range 20%-50% – see ELN-P05 QA/QC tables for details; <10xPQL - RPD are higher as the results approach PQL and the estimated measurement uncertainty will statistically increase.

Matrix Spikes, LCS and Surrogate recoveries: Generally 70-130% for inorganics/metals (not SPOCAS); 60-140% for organics/SPOCAS (+/-50% surrogates) and 10-140% for labile SVOCs (including labile surrogates), ultra trace organics and speciated phenols is acceptable.

In circumstances where no duplicate and/or sample spike has been reported at 1 in 10 and/or 1 in 20 samples respectively, the sample volume submitted was insufficient in order to satisfy laboratory QA/QC protocols.

When samples are received where certain analytes are outside of recommended technical holding times (THTs), the analysis has proceeded. Where analytes are on the verge of breaching THTs, every effort will be made to analyse within the THT or as soon as practicable.

Where sampling dates are not provided, Envirolab are not in a position to comment on the validity of the analysis where recommended technical holding times may have been breached.

Measurement Uncertainty estimates are available for most tests upon request.

Analysis of aqueous samples typically involves the extraction/digestion and/or analysis of the liquid phase only (i.e. NOT any settled sediment phase but inclusive of suspended particles if present), unless stipulated on the Envirolab COC and/or by correspondence. Notable exceptions include certain Physical Tests (pH/EC/BOD/COD/Apparent Colour etc.), Solids testing, total recoverable metals and PFAS where solids are included by default.

Samples for Microbiological analysis (not Amoeba forms) received outside of the 2-8°C temperature range do not meet the ideal cooling conditions as stated in AS2031-2012.

11 Attachment E – General Geotechnical Recommendations

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.

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

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

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

Subsurface Conditions - Changes

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

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

Subsurface Conditions - Site Anomalies

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

Report Use by Other Design Professionals

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

Subsurface Conditions – Geo-environmental Issues

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

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

Responsibility

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

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

Site Inspections

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

Soil Data

Explanation of Terms (1 of 3)

Definitions

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

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

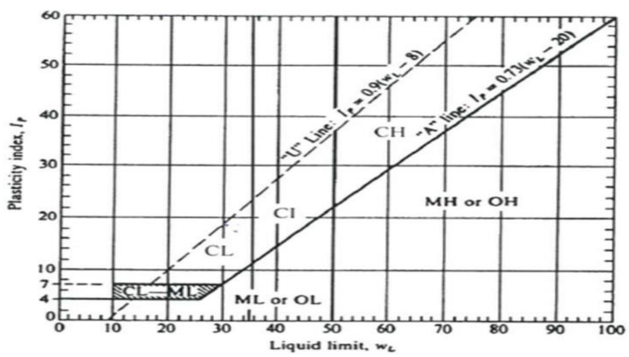
Particle Size

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

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

Plasticity Properties

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



Soil Moisture Condition

Coarse Grained (Granular) Soil:

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

Fine Grained (Cohesive) Soil:

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

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

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

Consistency of Cohesive Soils

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

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

Density of Granular Soils

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

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

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

Minor Components

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

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

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 or CLAY (OH or OL)
- CLAY (CL, CI or CH)
- Silty CLAY
- Sandy CLAY
- PEAT (Pt)
- Gravelly CLAY

OTHER

- FILL
- TALUS
- ASPHALT
- CONCRETE
- TOPSOIL

Unified Soil Classification Scheme (USCS)

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

		IDENTIFICATION PROCEDURES ON FRACTIONS < 0.2 MM					
		DRY STRENGTH (Crushing Characteristics)	**DILATANCY**	**TOUGHNESS**	**DESCRIPTION**	**USCS**	**Primary Name**
FINE GRAINED SOILS More than 35 % of material less than 63 mm is smaller than 0.075 mm	(A 0.075 mm particle is about the smallest particle visible to the naked eye)	None to Low	Quick to Slow	Low	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands or silt with low plasticity [2]	ML	SILT [3]
		Medium to High	None to Slow	Medium	Inorganic clays of low to medium plasticity, gravely clays, sandy clays, silty clays, lean clays	CL (or CI[4])	CLAY
		Low to Medium	Slow	Low	Organic slits and organic silty clays of low plasticity	OL	Organic SILT or CLAY
		Low to Medium	None to Slow	Low to Medium	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts	MH	SILT [3]
		High to Very High	None	High	Inorganic clays of high plasticity, fat clays	CH	CLAY
		Medium to High	None to Very Slow	Low to Medium	Organic clays of medium to high plasticity, organic silt of high plasticity	OH	Organic SILT or CLAY
HIGHLY ORGANIC SOILS		Readily identified by colour, odour, spongy feel and frequently by fibrous texture				Pt	PEAT

Notes:

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

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

martens consulting engineers

Symbols for Rock

SEDIMENTARY ROCK

- BRECCIA
- CONGLOMERATE
- CONGLOMERATIC SANDSTONE
- SANDSTONE/QUARTZITE
- SILTSTONE
- MUDSTONE/CLAYSTONE
- SHALE
- COAL
- LIMESTONE
- LITHIC TUFF

IGNEOUS ROCK

- GRANITE
- DOLERITE/BASALT

METAMORPHIC ROCK

- SLATE, PHYLLITE, SCHIST
- GNEISS
- METASANDSTONE
- METASILTSTONE
- METAMUDSTONE

Definitions

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

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

Degree of Weathering

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

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

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

Rock Strength

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

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

Degree of Fracturing

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

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

Rock Core Recovery

TCR = Total Core Recovery

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

SCR = Solid Core Recovery

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

RQD = Rock Quality Designation

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

Rock Strength Tests

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

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

● Uniaxial compressive strength (UCS) (MPa)

Defect Type Abbreviations and Descriptions

Defect Type (with inclination given)

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

Planarity

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

Roughness

Pol	Polished
Sl	Slickensided
Sm	Smooth
Ro	Rough
VR	Very rough

Thickness

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

Coating or Filling

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

Inclination

Inclination of defect is measured from perpendicular to and down the core axis.

Direction of defect is measured clockwise (looking down core) from magnetic north.

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

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

SOIL DESCRIPTION

Density

VL	Very loose
L	Loose
MD	Medium dense
D	Dense
VD	Very dense

Consistency

VS	Very soft
S	Soft
F	Firm
St	Stiff
VSt	Very stiff
H	Hard

Moisture

D	Dry
M	Moist
W	Wet
Wp	Plastic limit
Wl	Liquid limit

ROCK DESCRIPTION

Strength

VL	Very low
L	Low
M	Medium
H	High
VH	Very high
EH	Extremely high

Weathering

EW	Extremely weathered
HW	Highly weathered
MW	Moderately weathered
SW	Slightly weathered
FR	Fresh