APST
C/- APlus Design Group

Preliminary Geotechnical Assessment: 63 – 71 Waterloo Road, Macquarie Park, NSW

ENVIRONMENTAL

WATER

WASTEWATER

GEOTECHNICAL

CIVIL

PROJECT
MANAGEMENT

P1806577JR02V03
April 2020

Limitations Statement

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

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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1	Preliminary Geotechnical Assessment	Draft	20.06.2018	1E, 1H, 1P	1P		
2	Preliminary Geotechnical Assessment	Final	06.11.2018	1E, 1H, 1P	1P		
3	Preliminary Geotechnical Assessment	Final	21.04.2020	1E, 1H, 1P	1P		

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

Contents

1 Proposed Development and Investigation Scope

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

Table 1: Summary of proposed development and investigation scope.

Item	Details
Property Address	63 – 71 Waterloo Road, Macquarie Park, NSW ('the site')
Lot/DP	Lot 3, DP 1043041
LGA	City of Ryde Council (CRC)
Assessment Purpose	Preliminary geotechnical assessment to support a Stage 1 Development Application (DA) and assist structural design of the proposed development.
Site Area	20 337 m^2 (APlus Design Group, Feb 2020)
Proposed Development	The proposal plans (APlus Design Group, 2020) indicate that the development will include demolition of existing structures on site and construction of two new 65 m high commercial buildings (Building A and Building B), Building A has a lower ground floor level and five basement levels. Building B has a lower ground floor level and four basement levels. Maximum depth of excavation for each building is approximately 15 m. One new road will also be constructed along the north eastern site boundary to address CRC strategy. The proposed excavation for building A will be offset approximately 60 m, 15 m, 65 m and 20 m from the north western, south eastern, north eastern and south western site boundaries, respectively. The proposed excavation for building B will be offset approximately 14 m, 67 m, 75 m and 22 m from the north western, south eastern, north eastern and south western site boundaries, respectively.
Investigation Scope of Work	o Review of DBYD survey plans and underground service location on site. o A general site walkover survey. o Seven boreholes (BH101 to BH107) up to 6.6 mBGL (refer Attachment B for borehole logs, and associated explanatory notes in Attachment E). o Collection of soil and weathered rock samples for laboratory testing and for future reference. o Seven Dynamic Cone Penetrometer (DCP) tests (DCP101 to DCP107) up to 4.97 mBGL (refer DCP 'N' counts in Attachment C). Investigation locations are shown in Figure 1, Attachment A.
Laboratory Testing	Testing carried out by Resource Laboratories, National Association of Testing Authority (NATA) accredited laboratory comprised CBR testing of two collected bulk soil samples. Laboratory test certificates are provided in Attachment D.

2 General Site Details and Subsurface Conditions

General site details and investigation findings of subsurface conditions 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 slightly undulating terrain
Typical Slopes, Aspect, Elevation	The site generally has an easterly aspect with an overall grade of <5%. Site elevation ranges between approximately 53 m (northwest) and 59 m (southwest).
Expected Geology	The *Sydney 1:100,000 Geological Series Sheet 9130 (1983)* describes site geology as Ashfield Shale from the Wianamatta group, consisting of black to dark grey shale and laminate. The NSW Environment and Heritage eSPADE website identifies the site as having soils of the Glenorie soil landscapes consisting of shallow to moderately deep red podzolic soils on crests; moderately deep red and brown podzolic soils on upper slopes; deep yellow podzolic soils and greyed podzolic soils along drainage lines.
Existing Development	Site contains two separate commercial buildings as follows: o Commercial building occupied by Excelsia College at 69-71 Waterloo Road featuring performance theatre, classrooms, studios and offices. Asphalt and concrete carpark in the south east and north east. o Commercial building occupied by TPG Telecommunications at 65 Waterloo Road. The southern portion of the building is occupied by offices, with northern portion utilised as a warehouse for storage and dispatch of new electrical products. An asphalt carpark is located to the east and loading dock to the north.
Underground Services	An underground easement, 3050 mm wide, is marked on proposed plans (APlus Design Group, 2020) extending beneath the eastern portion of the site, bisecting the south eastern and north eastern site boundaries (Refer Figure 1, Attachment A). Review of the DBYD plans indicate that easement includes a 300 mm diameter vitrified clay sewer main pipe at depths of likely between 1.5 m and 3.0 m across the site. No more details of the easement were available at the time of reporting.
Vegetation	The site is mostly covered with asphalt and concrete driveways and carparks with some grass, bushes and mature trees located within garden beds and the landscaped area in the north and north eastern portion of the site.
Drainage	Via overland flow towards the east into existing easement.
Sub-surface Soil / Rock Units	Investigation revealed the following key subsurface units likely underlie concrete slab/ pavement within the investigated area: <u>Unit A</u>: Fill, comprising clay / silty clay / gravelly clay / sandy clay / gravelly silt / sand / sandy gravel with varying consistencies/densities, encountered to between approximately 0.5 mBGL and 2.0 mBGL. This unit has likely been placed for previous landscaping / levelling / easement construction / development purposes and considered to be "uncontrolled" for the purpose of this assessment. <u>Unit B:</u> Residual very stiff to hard and hard clay/silty clay up to between approximately 0.9 mBGL (BH104) and 5.5 mBGL (BH102).

Item	Comment
	Unit C: Weathered and inferred very low, very low to low and low strength shale/claystone/siltstone from depths of between approximately 0.9 mBGL and 3.0 mBGL. This unit was not encountered in BH102, being close to the existing site easement, up to termination depth of 5.50 mBGL. Investigations in BH105, BH106 and BH107 were terminated at 4.20 mBGL, 6.60 mBGL and 5.80 mBGL respectively, due to TC-bit refusal. In other boreholes, investigations terminated upon reaching target depths. For the purpose of this report, the rock below TC-bit refusal depths is assumed to be of medium strength with possible lower and / or higher strength bands, which should be confirmed / revised by further assessment, as necessary.
Groundwater	Groundwater inflow was not encountered during drilling of BH101 and BH103 to BH107 up to 6.6 mBGL. Groundwater inflow was encountered in BH102 at approximately 4.5 mBGL. Considering the location of this borehole (i.e. nearby easement), encountered groundwater is inferred to be potentially associated with the easement. Ephemeral perched groundwater may be encountered within the soil profile and/or at the soil/rock interface at times of, and following, heavy or extended periods of rainfall. 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 Preliminary Soil and Rock Properties

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

Table 3: Preliminary estimated soil and rock properties.

Layer [1]	$\gamma_{in\text{-}situ}$ [2] (kN/m³)	UCS [3] (MPa)	Cu [4] (kPa)	Ø' [5] (deg)	E' [6] (MPa)	K_s [7] (MPa/m)
FILL: Assumed uncontrolled	16-19	NA [8]	NA [8]	NA [8]	NA [8]	NA [8]
RESIDUAL SOIL: CLAY/Silty CLAY (very stiff to hard, dry)	17-18	0.3	150	NA [8]	20	20
RESIDUAL SOIL: CLAY/Silty CLAY (hard, dry)	19	0.4	200	NA [8]	30	33
WEATHERED ROCK: SHALE/CLAYSTONE/SILTSTONE (inferred very low to low strength)	22	0.5-1.0	NA [8]	28	100	75
WEATHERED ROCK: SHALE/CLAYSTONE/SILTSTONE (inferred low strength)	23	1.0-3.0	NA [8]	28	150	100
WEATHERED ROCK: SHALE/CLAYSTONE/SILTSTONE (inferred medium strength)	23	3.0-10.0	NA [8]	32	500	350

Notes:

1. Refer to borehole logs in Attachment B for material description details.
2. Inferred material in-situ unit weight, based on visual assessment (±10 %).
3. Average unconfined compressive strength of intact material (range provided for rock).
4. Undrained shear strength (± 5 kPa) estimate assuming normally consolidated clay in a dry condition.
5. Average effective internal friction angle (±2 ˚) estimate assuming drained conditions; may be dependent on rock defect conditions.
6. Effective elastic modulus (±10 %) estimate.
7. Modulus of subgrade reaction (vertical).
8. Not applicable.

3.2 Risk of Slope Instability

No evidence of former large-scale land instability was observed within the site and surrounding land during the site walkover.

A geotechnical hazard risk assessment in accordance with qualitative risk matrices provided in Section 7 of the AGS (2007) guidelines indicates the risk of potential slope instability, such as landslide or soil creep, to be low subject to the recommendations in this report.

The proposed excavations are likely to extend into the zone of influence of neighbouring properties to the north eastern site boundary.

Recommendations presented in this report are provided to mitigate risks associated with potential excavation instability during construction.

3.3 Geotechnical Recommendations

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

1. Excavation and support: Excavations in soils and extremely low to low strength rock must be temporarily and permanently supported to maintain excavation stability and limit potential adverse impacts on neighbouring properties or other infrastructure. Medium and / or higher strength rock, where encountered, may remain unsupported subject to confirmation on site by a geotechnical engineer. Appropriate support and / or excavation methodologies should be adopted by the excavation contractor and design engineer and approved by a geotechnical engineer.

   Excavation support may be provided by anchored soldier piles with shotcrete infill panels or contiguous concrete pile walls may be adopted particularly where higher wall stiffness is required to minimise deflections (e.g. near adjacent properties or existing services). Pile lengths may be terminated above bulk excavation level provided at least two rows of anchors are installed and the pile is socketed at least 2m into at least medium strength bedrock. Building floor slabs act as permanent lateral support for piles where these are adopted into the permanent works. Permission from neighbouring property owners is required if anchors extend beyond property boundaries.

2. Vibrations: If medium or higher strength rock is to be excavated using a rock hammer, vibration management will be required.

3. Earth Pressure Coefficients: Retaining wall design may adopt preliminary active, at rest and passive earth pressure coefficients of 0.4, 0.55 and 2.5, respectively.

4. Footings and Foundations: Shallow footings, such as pad and strip footings, or slab-on-ground founding on rock may be adopted as support for the proposed building. For foundations constructed on at least low strength Shale, an allowable bearing capacity of 1000kPa may be adopted, subject to an embedment depth of at least 0.5 m into the unit. Higher bearing capacities may be present

at bulk excavation subject to further investigation and testing. Individual pad footings and all footings within the building footprint should not span the interface between different foundation materials.

Deepened footings such as piles founding in at least medium strength rock may be considered to accommodate higher bearing pressures. Estimates of safe end bearing pressure and shaft friction for piles founding in medium strength rock are 1500 kPa and 250 kPa, respectively, subject to at least 0.5 m embedment into design unit. For uplift resistance, we recommend reducing allowable shaft friction by 50% and checking against 'piston' and 'cone' pull-out mechanisms in accordance with AS2159 (2009).

New lightly loaded structures with no required bulk excavation, if required, may be supported by piles founded on residual soil or underlying rock, depending on foundation level. These may be designed adopting allowable end bearing capacities of 200 kPa for very stiff to hard and 300 kPa for hard residual clay / silty clay.

Bearing capacity values should be confirmed by a geotechnical engineer on site during construction, as detailed in Section 5.2. Further testing is required if higher bearing capacities.

Retaining wall footings associated with bulk excavations should be founded below bulk basement excavation levels unless laterally supported by, for example, tie – back anchors or bracing.

5. <u>Drainage requirements</u>: Appropriate drainage measures should be provided upslope of the development and behind the retaining walls to divert overland flows and ephemeral perched water away from structures and discharge into council approved discharge points. Groundwater inflow, if encountered during excavation, is expected to be limited and can be managed by sump and pump methods.

   We recommend monitoring of flow during the early phases of excavation to assess potential long-term pumping requirements. Groundwater ingress should be monitored during excavation. Where high seepage inflows are encountered, long term dewatering is considered necessary. The anticipated volume of dewatering will determine if submission of a dewatering application and a subsequent dewatering license is required for the project.

   Where dewatering is required, we recommend a dewatering management plan be carried out to ensure dewatering works will not adversely impact adjacent or nearby structures and to assess water quality where discharging to local stormwater drain is required.

6. Site Classification: The site is classified as a "P" site in accordance with AS 2870 (2011).

4 Preliminary Pavement Thickness Design

4.1 Overview

A preliminary pavement thickness design was undertaken for the proposed internal access roads Australian Road Research Board (1989) *special report no. 41 (ARRB-SR41)* and Austroads (2012) *Guide to Pavement Technology Part 2 Pavement Structural Design (Austroads, 2012)*.

4.2 Design Parameters

An Equivalent Standard Axle (ESA) value of 3×10^5 was adopted for design of the proposed collector roads, based on Austroads (2012).

Two bulk soil samples were collected for CBR testing (Figure 1, Attachment A). Test results are summarised in Table 8. A laboratory test certificate is provided in Attachment D.

Table 8: Laboratory CBR test results.

Borehole	Sample Number	Material	Sample Depth (mBGL)	CBR [1] Value %
BH101	6577/CBR101	Silty CLAY	0.4 - 0.9	2.5
BH106	6577/CBR106	Silty CLAY	0.3 - 0.7	2.5

Given the limited laboratory test results, DCP-CBR correlations using Austroads (2012), the potential variation in soil moisture conditions, the likely cutting and filling required at the site as well as minimum acceptable CBR value based on *ARRB-SR41*, we have adopted a CBR value of 3 % for preliminary design purposes.

Subgrade stabilisation / replacement will be required where material of inferior quality is uncovered during excavation (i.e. CBR < 3 %).

Additional CBR testing is recommended to provide a better indication of subgrade conditions across pavement areas, considering final design levels, and / or provide statistical means to support a higher CBR design value.

4.3 Pavement Thickness

Table 9 presents recommended pavement material thicknesses for the proposed road.

Table 9: Preliminary pavement material thickness design for CBR 3 %.

Road Type	Total Thickness (mm)	Layer	Thickness (mm)	Materials
Category D – Collector Roads	490	Wearing Course	50	Primer + 10 mm one coat flush seal + 40 mm Asphalt Concrete (AC14)
		Base	150 [1]	DGB20
		Sub-base	290 [1]	CSS40 or DGS40

Notes:

[1] Based on Figure 8.4 of Ausroads 2012.

4.4 Subgrade Preparation

The subgrade is to be trimmed and compacted, following the removal of topsoil and other unsuitable materials such as root containing soils or uncontrolled fill, with density testing of the upper 300 mm layer at a rate of 1 test per 50 m of road length. Minimum relative density of subgrade shall be 100 % Maximum Dry Density (MDD) at a standard compactive effort within 2 % of optimum moisture content (OMC). Prior to placement of pavement material, the 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:

- Removal and replacement with approved granular fill under geotechnical engineer's direction.
- *In-situ* stabilisation with cement / lime or similar binding agent to a depth of at least 300 mm below finished level. Use of this method and extent will depend on the condition of material to be stabilised.

4.5 Subsoil Drainage

Surface and sub-soil drainage is to be provided in accordance with WSC requirements. Sub-surface drains are to be installed alongside the roads and generally extend minimum 600 mm below subgrade level.

4.6 Placement and Testing of Pavement Material

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

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

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

Compaction testing shall be undertaken by a NATA accredited laboratory in accordance with WSC, 2016. Testing should be carried out at a rate of 1 per 50 linear metres, or per 250 m^2, whichever is the greater, with a minimum of 2 tests in any one length. 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.

4.7 Fill Placement

Should filling be required to raise subgrade levels, the use of site-won excavated residual soils may be considered, subject to implementing stringent moisture conditioning and compaction controls, or mixing with lime to assist placement of medium or high plasticity clay, and testing. Alternatively, suitable granular fill, approved for use by a Geotechnical Engineer may be adopted. All earthworks specification is to be prepared by the supervising engineer and be implemented by the contractor.

5 Proposed Additional Works

5.1 Works Prior to Construction Certificate

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

1. Geotechnical investigation comprising at least three cored boreholes to approximately 18 m depth with point load testing of collected rock samples to assess rock strength.

2. Further CBR testing along proposed road alignments following completion of cut and fill design works.

3. Installation of groundwater monitoring wells to assess permanent groundwater levels, if required.

4. Where groundwater is encountered, a dewatering management plan should be carried out to satisfy Water NSW and Council requirements. This would include groundwater modelling to assess inflow rates and drawdown as well as water quality testing for discharge to stormwater, if required.

5. Review of the final design by a senior geotechnical engineer, if design carried out by a structural engineer, to confirm adequate consideration of the geotechnical risks and adoption of the recommendations provided in this report.

6. Review of preliminary pavement thickness design following receipt of pavement design specifications from City of Ryde Council.

5.2 Construction Monitoring and Inspections

We recommend the following is inspected and monitored during construction of the project (Table 4).

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

Scope of Works	Frequency/Duration	Who to Complete
Inspect excavation retention (shoring, retaining wall) installations, and of batters, monitor associated performance to assess need for additional support requirements.	Daily / As required [2]	Builder / MA [1]
Monitor groundwater seepage from excavation faces, if encountered, to assess stability of exposed materials and additional drainage requirements.	When encountered	Builder / MA [1]

Scope of Works	Frequency/Duration	Who to Complete
Monitor excavation-induced vibrations, if medium or higher strength rock is to be excavated.	Daily at on-set of excavation and as agreed thereafter [2]	MA [1]
Monitor settlement and lateral deflection of retaining structures to identify potential excavation impacts on neighbouring properties.	Daily at on-set of excavation and as agreed thereafter	Builder / MA [1]
Inspect exposed material at foundation / subgrade level to verify suitability as foundation / lateral support / subgrade.	Prior to reinforcement set-up and concrete placement	MA [1]
Monitor sedimentation downslope of excavated areas.	During and after rainfall events	Builder
Monitor sediment and erosion control structures to assess adequacy and for removal of built up spoil.	After rainfall events	Builder

Notes:

[1] MA = Martens and Associates engineer

[2] MA inspection frequency to be determined based on initial inspection findings in line with construction program.

6 References

APlus Design Group (2020) Architectural Drawings, Project No. A17146, Drawing Nos: A1.02, A2.03, A3.01 – A3.13, A5.02 (all revision A), dated 21 Feb 2020.

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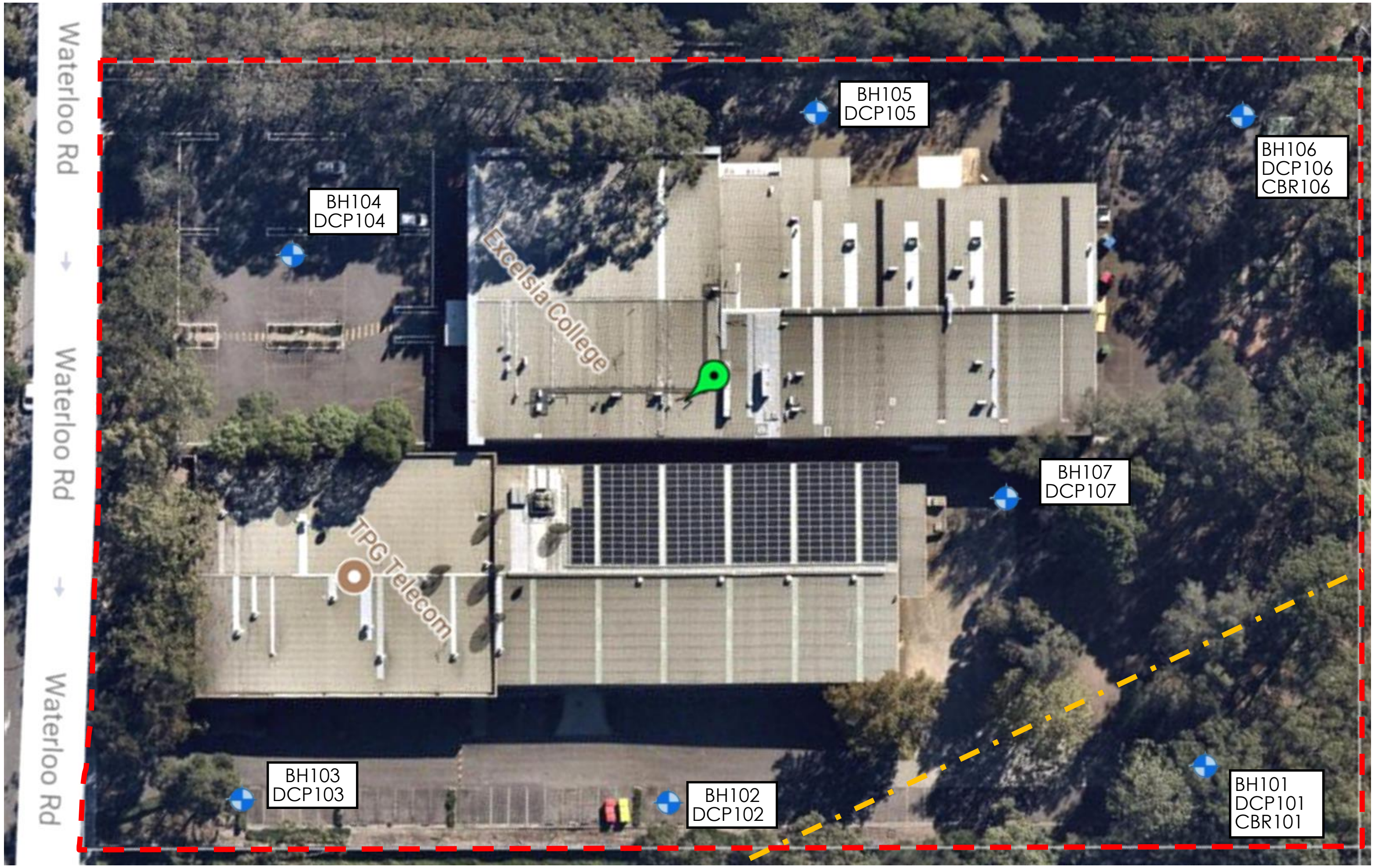
Standards Australia Limited (1997) AS 1289.6.3.2:1997, *Determination of the penetration resistance of a soil – 9kg dynamic cone penetrometer test*, SAI Global Limited.

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

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

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

7 Attachment A – Site layout and Geotechnical Testing Plan



martens

Key:

Approximate borehole, DCP test, and CBR Sampling location.

Indicative easement alignment

Approximate site boundary

Martens & Associates Pty Ltd ABN 85 070 240 890		Environment | Water | Wastewater | Geotechnical | Civil | Management	
Drawn:	HD	**GEOTECHNICAL TESTING PLAN** **63 – 71 Waterloo Road, Macquarie Park, NSW** (Source: Nearmap, 2018)	Drawing No: **FIGURE 1**
Approved:	HN		
Date:	19.06.2018		
Scale:	NA		Project Number: P1806577JR02V01

8 Attachment B – Test Borehole Logs

CLIENT	APST-c/- APlus Design Group	COMMENCED	05/06/2017	COMPLETED	05/06/2017	**REF BH101**
PROJECT	Preliminary Geotechnical Assessment	LOGGED	HD	CHECKED	HN	Sheet 1 OF 1
SITE	63-71 Waterloo Road, Macquarie Park, NSW	GEOLOGY	Ashfield Shale	VEGETATION	Grass/dirt	PROJECT NO. P1806577

EQUIPMENT	4WD truck-mounted hydraulic drill rig	EASTING		RL SURFACE	m	DATUM	Existing Ground Level
EXCAVATION DIMENSIONS	Ø200 mm x 1.00 m depth	NORTHING		ASPECT	Southwest	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-M	Not Encountered		0.30	6577/BH101/0.1-0.2/S/1 D 0.10 m			ML	FILL: Gravelly SILT; low plasticity; dark brown; with fine grained sand.	M (<<PL)	D	FILL
				1.00	6577/BH101/0.3-0.5/S/1 D 0.30 m 6577/BH101/0.4-0.9/B/1 CBR 0.40 m			CL-CI	FILL: Silty CLAY; low to medium plasticity; dark brown; trace fine ironstone gravels; trace fine grained sand.		VSt to H	
			1						Hole Terminated at 1.00 m (Target depth reached)			
			2									
			3									
			4									
			5									
			6									
			7									

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

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Engineering Log - BOREHOLE

MARTENS 2.00 LIB.GLB Log MARTENS BOREHOLE P1806577BH101-BH107V01.GPJ <<DrawingFile>> 21/06/2018 15:55 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	APST-c/- APlus Design Group	COMMENCED	05/06/2017	COMPLETED	05/06/2017	REF BH102
PROJECT	Preliminary Geotechnical Assessment	LOGGED	HD	CHECKED	HN	Sheet 1 OF 1
SITE	63-71 Waterloo Road, Macquarie Park, NSW	GEOLOGY	Ashfield Shale	VEGETATION	None	PROJECT NO. P1806577
EQUIPMENT	4WD truck-mounted hydraulic drill rig	EASTING		RL SURFACE	m	DATUM: Existing Ground Level
EXCAVATION DIMENSIONS	Ø100 mm x 5.50 m depth	NORTHING		ASPECT	Northeast	SLOPE: <5%

METHOD	PENETRATION RESISTANCE	WATER	DEPTH (metres)	DEPTH RL	SAMPLE OR FIELD TEST	RECOVERED	GRAPHIC LOG	USCS / ASCS CLASSIFICATION	SOIL/ROCK MATERIAL DESCRIPTION	MOISTURE CONDITION	CONSISTENCY DENSITY	STRUCTURE AND ADDITIONAL OBSERVATIONS
AD/V	L								FILL: ASHPHALT			PAVEMENT
				0.20	6577/BH102/0.2-0.3/S/1 D 0.20 m			CL-CI	FILL: Gravelly CLAY; low to medium plasticity; dark grey; with fine to medium grained sand.	M (<<PL)	S	FILL
								CL-CI	FILL: Sandy CLAY, low to medium plasticity; yellow brown.			
				0.60	6577/BH102/0.6-0.8/S/1 D 0.60 m			SP	FILL: SAND; yellow; fine to medium grained; trace clay; trace shale gravels.	D	L and MD	
AD/T	H		1	1.00				CL-CI	FILL: Gravelly CLAY; low to medium plasticity; dark grey; (gravels >20mm).			1.00: V-bit refusal on inferred gravels within soil profile.
				1.20				CI	FILL: Silty CLAY; medium plasticity; orange, trace pale grey and red; with medium to coarse ironstone gravels; trace roadbase gravels.			
	L-M				6577/BH102/1.8-2.0/S/1 D 1.80 m							
			2	2.00				CI-CH	Silty CLAY; medium to high plasticity; pale grey, trace red; trace ironstone gravels.	M (<<PL)	VSt	RESIDUAL SOIL
			3		6577/BH102/3.0-3.2/S/1 D 3.00 m							
	M		4							M (<PL)	H	
		05/06/18		4.50					Pale-grey, red and red-brown.			
			5		6577/BH102/4.8-5.0/S/1 D 4.80 m					W		
				5.30					Pale-grey with dark red.			
				5.50	6577/BH102/5.4-5.5/S/1 D 5.40 m				Hole Terminated at 5.50 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	APST-c/- APlus Design Group	COMMENCED	05/06/2017	COMPLETED	05/06/2017	REF BH103	
PROJECT	Preliminary Geotechnical Assessment	LOGGED	HD	CHECKED	HN	Sheet 1 OF 1	
SITE	63-71 Waterloo Road, Macquarie Park, NSW	GEOLOGY	Ashfield Shale	VEGETATION	None	PROJECT NO. P1806577	
EQUIPMENT	4WD truck-mounted hydraulic drill rig	EASTING		RL SURFACE	m	DATUM	Existing Ground Level
EXCAVATION DIMENSIONS	Ø100 mm x 4.00 m depth	NORTHING		ASPECT	Northeast	SLOPE	<5%

METHOD	PENETRATION RESISTANCE	WATER	DEPTH (metres)	DEPTH RL	SAMPLE OR FIELD TEST	RECOVERED	GRAPHIC LOG	USCS / ASCS CLASSIFICATION	SOIL/ROCK MATERIAL DESCRIPTION	MOISTURE CONDITION	CONSISTENCY DENSITY	STRUCTURE AND ADDITIONAL OBSERVATIONS
AD/V	L-M	Not Encountered							FILL: ASHPHALT	M (<<PL)		PAVEMENT
				0.50	6577/BH103/0.3-0.4/S/1 D 0.30 m			CL-CI	FILL: CLAY; low to medium plasticity; red, pale grey and orange; with gravels.		St	FILL
				0.80	6577/BH103/0.6-0.8/S/1 D 0.60 m			CL-CI	FILL: Silty CLAY; low to medium plasticity; orange; trace fine grained sand; trace ironstone gravels.			
			1	1.50	6577/BH103/1.1-1.4/S/1 D 1.10 m			CI-CH	CLAY; medium to high plasticity; red, orange and grey-brown; trace fine ironstone gravels; trace fine grained sand.		VSt - H	RESIDUAL SOIL
AD/T	L		2	2.00	6577/BH103/1.6-1.7/S/1 D 1.60 m			CI-CH	CLAY; medium to high plasticity; red, pale grey, trace orange; trace medium to coarse ironstone gravels; trace fine grained sand, inferred hard.			1.50: V-bit refusal on inferred ironstone band/gravels.
				3.00	6577/BH103/2.0-2.4/S/1 D 2.00 m				Pale-grey with red; no ironstone gravels.		H	
	L-M		3	4.00	6577/BH103/3.4/R/1 D 3.40 m 6577/BH103/3.7-4.0/R/1 D 3.70 m				SHALE/CLAYSTONE; red, dark grey and pale grey; highly weathered; inferred very low strength.			WEATHERED ROCK
			4						Hole Terminated at 4.00 m (Target depth reached)			
			5									
			6									
			7									

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

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Engineering Log - BOREHOLE

CLIENT	APST-c/- APlus Design Group	COMMENCED	05/06/2017	COMPLETED	05/06/2017	REF	BH104
PROJECT	Preliminary Geotechnical Assessment	LOGGED	HD	CHECKED	HN	Sheet	1 OF 1
SITE	63-71 Waterloo Road, Macquarie Park, NSW	GEOLOGY	Ashfield Shale	VEGETATION	None	PROJECT NO. P1806577	
EQUIPMENT	4WD truck-mounted hydraulic drill rig	EASTING		RL SURFACE	m	DATUM	Existing Ground Level
EXCAVATION DIMENSIONS	Ø100 mm x 5.50 m depth	NORTHING		ASPECT	Southeast	SLOPE	4-5%

METHOD	PENETRATION RESISTANCE	WATER	DEPTH (metres)	DEPTH RL	SAMPLE OR FIELD TEST	RECOVERED	GRAPHIC LOG	USCS / ASCS CLASSIFICATION	SOIL/ROCK MATERIAL DESCRIPTION	MOISTURE CONDITION	CONSISTENCY DENSITY	STRUCTURE AND ADDITIONAL OBSERVATIONS
AD/V	M	Not Encountered							FILL: ASHPHALT.			PAVEMENT
				0.50	6577/BH104/0.2-0.3/S/1 D 0.20 m			GP	FILL: Sandy GRAVEL; fine to coarse grain, grey and grey-brown, with blue metal.	D	D	FILL
				0.90	6577/BH104/0.7-0.8/S/1 D 0.70 m			CI-CH	CLAY; medium to high plasticity; pale grey and red-orange; trace ironstone gravels.	M (<<PL)	H	RESIDUAL SOIL
AD/T	H		1						CLAYSTONE/SHALE; pale grey, red-brown, with red-orange and orange-brown; distinctly weathered; inferred low strength.			WEATHERED ROCK 0.90: V-bit refusal.
			2		6577/BH104/2.5-3.0/R/1 D 2.50 m							
			3									
			4	4.00					Dark brown.			
			5	5.50	6577/BH104/5.0-5.5/R/1 D 5.00 m							
			6						Hole Terminated at 5.50 m (Target depth reached)			
			7									

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Engineering Log - BOREHOLE

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CLIENT	APST-c/- APlus Design Group	COMMENCED	05/06/2017	COMPLETED	05/06/2017	REF BH105
PROJECT	Preliminary Geotechnical Assessment	LOGGED	HD	CHECKED	HN	Sheet 1 OF 1
SITE	63-71 Waterloo Road, Macquarie Park, NSW	GEOLOGY	Ashfield Shale	VEGETATION	None	PROJECT NO. P1806577
EQUIPMENT	4WD truck-mounted hydraulic drill rig	EASTING		RL SURFACE	m	DATUM: Existing Ground Level
EXCAVATION DIMENSIONS	Ø100 mm x 4.20 m depth	NORTHING		ASPECT	N/A	SLOPE: <1%

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
DT				0.20					CONCRETE SLAB.			PAVEMENT
AD/T	M	Not Encountered			6577/BH105/0.2-0.4/S/1 D 0.20 m			SP	FILL: SAND; fine to coarse grained; red-brown; trace fine to coarse subrounded sandstone gravels; trace clay, inferred medium dense to dense.	D	MD - D	FILL
			1	0.70	6577/BH105/0.7-0.8/S/1 D 0.70 m			CL-CI	FILL: Sandy CLAY; low to medium plasticity; brown and red-brown; trace teracota/brick fragments.	M (<<PL)	VSt - H	
	H		2	1.20	6577/BH105/1.2-1.4/R/1 D 1.20 m				SHALE/SILTSTONE; pale grey and orange-brown; distinctly weathered; inferred low strength.			WEATHERED ROCK
				1.80					Pale grey.			
			3	2.80	6577/BH105/3.0-3.5/R/1 D 3.00 m				Dark grey.			
			4	3.80					Ironstone bands.			
			5, 6, 7	4.20					Hole Terminated at 4.20 m			4.20: TC-bit refusal on inferred medium strength shale.

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CLIENT	APST-c/- APlus Design Group	COMMENCED	05/06/2017	COMPLETED	05/06/2017	REF BH106
PROJECT	Preliminary Geotechnical Assessment	LOGGED	HD	CHECKED	HN	Sheet 1 OF 1
SITE	63-71 Waterloo Road, Macquarie Park, NSW	GEOLOGY	Ashfield Shale	VEGETATION	None	PROJECT NO. P1806577
EQUIPMENT	4WD truck-mounted hydraulic drill rig	EASTING		RL SURFACE	m	DATUM: Existing Ground Level
EXCAVATION DIMENSIONS	Ø100 mm x 6.60 m depth	NORTHING		ASPECT	Southeast	SLOPE: <5%

METHOD	PENETRATION RESISTANCE	WATER	DEPTH (metres)	DEPTH RL	SAMPLE OR FIELD TEST	RECOVERED	GRAPHIC LOG	USCS / ASCS CLASSIFICATION	SOIL/ROCK MATERIAL DESCRIPTION	MOISTURE CONDITION	CONSISTENCY DENSITY	STRUCTURE AND ADDITIONAL OBSERVATIONS
AD/T	M	Not Encountered							FILL: ASHPHALT			PAVEMENT
				0.20	6557/BH106/0.3-0.7/B/1 CBR 0.30 m			SP	FILL: SAND; fine to coarsed grained; yellow-brown; trace fine subrounded gravels, inferred medium dense.	D	MD	FILL
								CI-CH	FILL: CLAY; medium to high plasticity; red, pale-grey and orange-red, trace black; trace fine to medium grained sand.			
				0.50	6557/BH106/0.8-1.1/S/1 D 0.80 m			CI-CH	CLAY; medium to high plasticity; red, pale-grey and orange; trace fine to medium grained sand.		VSt	RESIDUAL SOIL
			1	1.20	6557/BH106/1.4-1.7/S/1 D 1.40 m			CL-CI	Silty CLAY; low to medium plasticity; brown; trace fine grained sand; trace fine gravels, inferred very stiff to hard.	M (<<PL)		
			2	1.70	6557/BH106/2.0-2.3/S/1 D 2.00 m			CI-CH	CLAY; medium to high plasticity; red, pale grey and orange, inferred very stiff to hard.		VSt - H	
			3	3.00	6557/BH106/4.0-4.2/R/1 D 4.00 m; 6557/BH106/6.0-6.5/R/1 D 6.00 m				SHALE; pale grey, trace red; distinctly weathered; inferred very low to low strength.			WEATHERED ROCK
			4, 5, 6, 7	6.60					Hole Terminated at 6.60 m			6.60: TC-bit refusal on inferred medium strength shale.

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Engineering Log - BOREHOLE

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CLIENT	APST-c/- APlus Design Group	COMMENCED	05/06/2017	COMPLETED	05/06/2017	REF BH107
PROJECT	Preliminary Geotechnical Assessment	LOGGED	HD	CHECKED	HN	Sheet 1 OF 1
SITE	63-71 Waterloo Road, Macquarie Park, NSW	GEOLOGY	Ashfield Shale	VEGETATION	None	PROJECT NO. P1806577
EQUIPMENT	4WD truck-mounted hydraulic drill rig	EASTING		RL SURFACE	m	DATUM: Existing Ground Level
EXCAVATION DIMENSIONS	Ø100 mm x 5.80 m depth	NORTHING		ASPECT	East	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-H	Not Encountered							FILL: ASPHALT.			PAVEMENT
				0.70	6577/BH107/0.3-0.5/S/1 D 0.30 m 6577/BH107/0.6-0.7/S/1 D 0.60 m			SP	FILL: SAND; fine to medium grained; orange and yellow-brown; with fine to coarse subrounded sandstone gravels.	D	D	FILL
			1 2	2.30	6577/BH107/2.0-2.2/S/1 D 2.00 m			CI	CLAY; medium plasticity; red, pale grey and brown; trace fine to coarse ironstone gravels.	M (<<PL)	H	RESIDUAL SOIL
AD/T	L-M		3 4	4.00	6577/BH107/3.5-3.8/R/1 D 3.50 m				SHALE/SILTSTONE; brown and pale grey; distinctly weathered; inferred very low to low strength; trace ironstone.			WEATHERED ROCK 2.30: V-bit refusal.
			5	5.00	6577/BH107/4.0-4.4/R/1 D 4.00 m				SILTSTONE; pale grey; distinctly weathered; inferred very low to low strength; trace ironstone.			
	M			5.80	6577/BH107/5.6-5.8/R/1 D 5.60 m				SHALE; pale grey and dark grey; distinctly weathered; inferred very low to low strength.			
			6 7						Hole Terminated at 5.80 m			5.80: TC-bit refusal on inferred medium strength shale.

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9 Attachment C – DCP 'N' Counts

Dynamic Cone Penetrometer Test Log Summary

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Site	63 - 71 Waterloo Road, Macquarie Park, NSW	**DCP Group Reference**	P1806577JS01V01
Client	APST C/- APlus Design Group	**Log Date**	05.06.2018
Logged by	HD/MV		
Checked by	HN		
Comments	DCPs commenced 50 mm BGL. Where required DCPs were commenced following removal of concrete/ashphalt and fill.		

TEST DATA

Depth Interval (m)	DCP101	DCP102	DCP103	DCP104	DCP105	DCP106	DCP107	
0.15	12	Asphalt/Fill	Asphalt	Asphalt	Ashphalt	Asphalt/Fill	16	
0.30	14		7	22	Fill		33	
0.45	19	1	19	20		16	40+	
0.60	27	1	24	23	7	15	Fill	
0.75	16	1	36	33	20	13		
0.90	17	3	30	31	Bounce @ 0.80 mBGL	Bounce @ 0.8 mBGL		
1.05	17	5	26	Bounce @ 0.95 mBGL			18	
1.20	14	16	24				20	
1.35	12	9	20/120mm				20	
1.50	22	10	Bounce @ 1.37 mBGL				28	
1.65	30	8					29	
1.80	25	13					22	
1.95	35	8					23	
2.10	40+	13					17	
2.25	Terminated @ 2.15 m due to high 'N' counts	13					16	
2.40		17					25	
2.55		15					30	
2.70		16					Bounce @ 2.6 mBGL	
2.85		18						
3.00		15						
3.15		17						
3.30		20						
3.45		23						
3.60		31						
3.75		21						
3.90		21						
4.05		23						
4.20		21						
4.35		23						
4.50		26						
4.65		25						
4.80		25						
4.95		36/120 mm						
5.10		Terminated @4.97 m due to high 'N' counts						
5.25								
5.40								

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: P1806577
Location: A+Design Group

Job number: 18-0076
Report number: 1

California Bearing Ratio

Sampling method: Samples tested as received

Test method(s): AS 1289.1.1, 2.1.1, 5.1.1, 6.1.1

	Results			
Laboratory sample no.	15394	15395		
Customer sample no.	6577/BH101/ 0.4-0.9	6577/BH106/ 0.3-0.7		
Date sampled	05/06/2018	05/06/2018		
Material description	silty CLAY, with sand and gravel, brown	silty CLAY, trace of gravel and sand, brown mottled red/grey		
Maximum dry density (t/m^3)	1.78	1.70		
Optimum moisture content (%)	16.1	18.9		
Field moisture content (%)	n/a	n/a		
Oversize retained on 19.0mm sieve (%)	1	1		
Minimum curing time (hours)	48	96		
Dry density before soak (t/m^3)	1.74	1.65		
Dry density after soak (t/m^3)	1.72	1.61		
Moisture content before soak (%)	16.0	19.4		
Moisture content after soak (%)	19.5	23.3		
Moisture content after test - top 30mm (%)	21.3	24.8		
Moisture content after test - remaining depth (%)	18.4	21.4		
Density ratio before soaking (%)	98.0	97.5		
Moisture ratio before soaking (%)	99.5	103.0		
Period of soaking (days)	4	4		
Compactive effort	Standard	Standard		
Mass of surcharge applied (kg)	4.5	4.5		
Swell after soaking (%)	1.5	2.5		
Penetration (mm)	2.5	5.0		
CBR Value (%)	**2.5**	**2.5**		

Notes: Specified LDR: 98 ±1%

Method of establishing plasticity level - Visual / tactile

Approved Signatory: C. Greely

Date: 26/06/2018

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 E – Notes About This Report

Important Information About Your Report (1 of 2)

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

Engineering Reports - Limitations

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

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

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

Engineering Reports – Project Specific Criteria

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

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

Engineering Reports – Recommendations

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

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

Engineering Reports – Use for Tendering Purposes

Where information obtained from investigations is provided for tendering purposes, Martens recommend that all information, including the written report and discussion, be made available. In circumstances where the discussion or comments section is not relevant to the contractual situation, it may be appropriate to prepare a specially edited document.

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

Engineering Reports – Data

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

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

Engineering Reports – Other Projects

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

Subsurface Conditions - General

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

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

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

- The actions of contractors responding to commercial pressures.

- Actual conditions differing somewhat from those inferred to exist, because no professional, no matter how qualified, can reveal precisely what is hidden by earth, rock and time.

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

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

Subsurface Conditions - Changes

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

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

Subsurface Conditions - Site Anomalies

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

Report Use by Other Design Professionals

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

Subsurface Conditions – Geo-environmental Issues

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

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

Responsibility

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

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

Site Inspections

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

Soil Data

Explanation of Terms (1 of 3)

Definitions

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

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

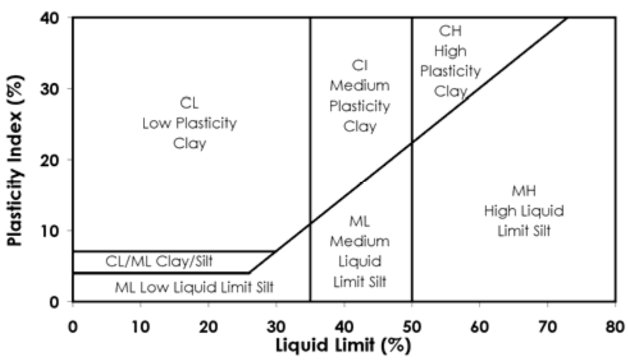
Particle Size

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

Division	Subdivision	Size (mm)
BOULDERS		>200
COBBLES		63 to 200
GRAVEL	Coarse	20 to 63
	Medium	6 to 20
	Fine	2.36 to 6
SAND	Coarse	0.6 to 2.36
	Medium	0.2 to 0.6
	Fine	0.075 to 0.2
SILT		0.002 to 0.075
CLAY		< 0.002

Plasticity Properties

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



Moisture Condition

Dry — Looks and feels dry. Cohesive and cemented soils are hard, friable or powdery. Uncemented granular soils run freely through hands.

Moist — Soil feels cool and damp and is darkened in colour. Cohesive soils can be moulded. Granular soils tend to cohere.

Wet — As for moist but with free water forming on hands when handled.

Consistency of Cohesive Soils

Cohesive soils refer to predominantly clay materials.

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

Density of Granular Soils

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

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

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

Minor Components

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

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

Soil Data

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Symbols for Soils and Other

SOILS

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

OTHER

- FILL
- TALUS
- ASPHALT
- CONCRETE

Unified Soil Classification Scheme (USCS)

<table>
<tr><th colspan="6">FIELD IDENTIFICATION PROCEDURES
(Excluding particles larger than 63 mm and basing fractions on estimated mass)</th><th>USCS</th><th>Primary Name</th></tr>
<tr><td rowspan="8">COARSE GRAINED SOILS
More than 50 % of material less than 63 mm is larger than 0.075 mm</td><td rowspan="16">(A 0.075 mm particle is about the smallest particle visible to the naked eye)</td><td rowspan="4">GRAVELS
More than half of coarse fraction is larger than 2.0 mm.</td><td rowspan="2">CLEAN GRAVELS (Little or no fines)</td><td colspan="2">Wide range in grain size and substantial amounts of all intermediate particle sizes.</td><td>GW</td><td>Gravel</td></tr>
<tr><td colspan="2">Predominantly one size or a range of sizes with more intermediate sizes missing</td><td>GP</td><td>Gravel</td></tr>
<tr><td rowspan="2">GRAVELS WITH FINES (Appreciable amount of fines)</td><td colspan="2">Non-plastic fines (for identification procedures see ML below)</td><td>GM</td><td>Silty Gravel</td></tr>
<tr><td colspan="2">Plastic fines (for identification procedures see CL below)</td><td>GC</td><td>Clayey Gravel</td></tr>
<tr><td rowspan="4">SANDS
More than half of coarse fraction is smaller than 2.0 mm</td><td rowspan="2">CLEAN SANDS (Little or no fines)</td><td colspan="2">Wide range in grain sizes and substantial amounts of intermediate sizes missing.</td><td>SW</td><td>Sand</td></tr>
<tr><td colspan="2">Predominantly one size or a range of sizes with some intermediate sizes missing</td><td>SP</td><td>Sand</td></tr>
<tr><td rowspan="2">SANDS WITH FINES (Appreciable amount of fines)</td><td colspan="2">Non-plastic fines (for identification procedures see ML below)</td><td>SM</td><td>Silty Sand</td></tr>
<tr><td colspan="2">Plastic fines (for identification procedures see CL below)</td><td>SC</td><td>Clayey Sand</td></tr>
<tr><td rowspan="8">FINE GRAINED SOILS
More than 50 % of material less than 63 mm is smaller than 0.075 mm</td><td colspan="6">IDENTIFICATION PROCEDURES ON FRACTIONS < 0.2 MM</td></tr>
<tr><th>DRY STRENGTH (Crushing Characteristics)</th><th>DILATANCY</th><th>TOUGHNESS</th><th>DESCRIPTION</th><th>USCS</th><th>Primary Name</th></tr>
<tr><td>None to Low</td><td>Quick to Slow</td><td>None</td><td>Inorganic silts and very fine sands, rock flour, silty or clayey fine sands with slight plasticity</td><td>ML</td><td>Silt</td></tr>
<tr><td>Medium to High</td><td>None</td><td>Medium</td><td>Inorganic clays of low to medium plasticity [1], gravely clays, sandy clays, silty clays, lean clays</td><td>CL [2]</td><td>Clay</td></tr>
<tr><td>Low to Medium</td><td>Slow to Very Slow</td><td>Low</td><td>Organic slits and organic silty clays of low plasticity</td><td>OL</td><td>Organic Silt</td></tr>
<tr><td>Low to Medium</td><td>Slow to Very Slow</td><td>Low to Medium</td><td>Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts</td><td>MH</td><td>Silt</td></tr>
<tr><td>High</td><td>None</td><td>High</td><td>Inorganic clays of high plasticity, fat clays</td><td>CH</td><td>Clay</td></tr>
<tr><td>Medium to High</td><td>None</td><td>Low to Medium</td><td>Organic clays of medium to high plasticity</td><td>OH</td><td>Organic Silt</td></tr>
<tr><td colspan="2">HIGHLY ORGANIC SOILS</td><td colspan="4">Readily identified by colour, odour, spongy feel and frequently by fibrous texture</td><td>Pt</td><td>Peat</td></tr>
<tr><td colspan="8">Notes:
1. Low Plasticity – Liquid Limit W_L < 35 % Medium Plasticity – Liquid limit W_L 35 to 60 % High Plasticity - Liquid limit W_L > 60 %.
2. CI may be adopted for clay of medium plasticity to distinguish from clay of low plasticity.</td></tr>
</table>

Soil Agricultural Classification Scheme

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

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

Rock Data

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Symbols for Rock

SEDIMENTARY ROCK

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

IGNEOUS ROCK

- GRANITE
- DOLERITE/BASALT

METAMORPHIC ROCK

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

Definitions

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

Rock Substance — In geotechnical engineering terms, rock substance is any naturally occurring aggregate of minerals and organic matter which cannot be disintegrated or remoulded by hand in air or water. Other material is described using soil descriptive terms. Rock substance is effectively homogeneous and may be isotropic or anisotropic.

Rock Defect — Discontinuity or break in the continuity of a substance or substances.

Rock Mass — Any body of material which is not effectively homogeneous. It can consist of two or more substances without defects, or one or more substances with one or more defects.

Degree of Weathering

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

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

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

Rock Strength

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

Term	Is (50) MPa	Field Guide	Symbol
Very low	>0.03 ≤0.1	May be crumbled in the hand. Sandstone is 'sugary' and friable.	VL
Low	>0.1 ≤0.3	A piece of core 150mm long x 50mm diameter may be broken by hand and easily scored with a knife. Sharp edges of core may be friable and break during handling.	L
Medium	>0.3 ≤1.0	A piece of core 150mm long x 50mm diameter can be broken by hand with considerable difficulty. Readily scored with a knife.	M
High	>1 ≤3	A piece of core 150mm long x 50mm diameter cannot be broken by unaided hands, can be slightly scratched or scored with a knife.	H
Very high	>3 ≤10	A piece of core 150mm long x 50mm diameter may be broken readily with hand held hammer. Cannot be scratched with pen knife.	VH
Extremely high	>10	A piece of core 150mm long x 50mm diameter is difficult to break with hand held hammer. Rings when struck with a hammer.	EH

Degree of Fracturing

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

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

Rock Core Recovery

TCR = Total Core Recovery

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

SCR = Solid Core Recovery

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

RQD = Rock Quality Designation

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

Rock Strength Tests

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

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

● Unconfined compressive strength (UCS) (MPa)

Defect Type Abbreviations and Descriptions

Defect Type (with inclination given)

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

Planarity

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

Roughness

Pol	Polished
Sl	Slickensided
Sm	Smooth
Ro	Rough
VR	Very rough

Thickness

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

Coating or Filling

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

Inclination

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

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

Sampling

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

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

Undisturbed samples may be taken by pushing a thin-walled sampling tube, e.g. U_{50} (50 mm internal diameter thin walled tube), into soils and withdrawing a soil sample in a relatively undisturbed state. Such samples yield information on structure and strength and are necessary for laboratory determination of shear strength and compressibility. Undisturbed sampling is generally effective only in cohesive soils. Other sampling methods may be used. Details of the type and method of sampling are given in the report.

Drilling / Excavation Methods

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

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

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

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

Large Diameter Auger (e.g. Pengo) - the hole is advanced by a rotating plate or short spiral auger, generally 300 mm or larger in diameter. The cuttings are returned to the surface at intervals (generally of not more than 0.5 m) and are disturbed but usually unchanged in moisture content. Identification of soil strata is generally much more reliable than with continuous spiral flight augers, and is usually supplemented by occasional undisturbed tube sampling.

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

Continuous Spiral Flight Augers - the hole is advanced using 90 - 115 mm diameter continuous spiral flight augers, which are withdrawn at intervals to allow sampling or in-situ testing. This is a relatively economical means of drilling in clays and in sands above the water table. Samples are returned to the surface or, or may be collected after withdrawal of the auger flights, but they are very disturbed and may be contaminated. Information from the drilling (as distinct from specific sampling by SPTs or undisturbed samples) is of relatively lower reliability, due to remoulding, contamination or softening of samples by ground water.

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

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

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

In-situ Testing and Interpretation

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

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

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

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

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

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

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

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

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

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

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

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

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

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

Standard Penetration Testing (SPT)

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

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

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

as 4, 6, 7
N = 13

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

as 15, 30/40 mm.

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

Dynamic Cone (Hand) Penetrometers

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

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

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

Pocket Penetrometers

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

$q_u = 2 \times C_u$.

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

Test Pit / Borehole Logs

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

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

Laboratory Testing

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

Ground Water

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

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

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

Test, Drill and Excavation Methods

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

s	Water level at date shown	v	Partial water loss
w	Water inflow	t	Complete water loss

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

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

PENETRATION / EXCAVATION RESISTANCE

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

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

SAMPLING

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

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

TESTING

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

Notes:

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

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

SOIL DESCRIPTION

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

ROCK DESCRIPTION

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