

# Preliminary Geotechnical Assessment

2-30 Tempus Street, Rouse Hill, NSW

Final Report

P2410429JR03V02

March 2025

Prepared for Freecity Rouse Hill Development Pty Ltd

## Project Details

<b>Report Title</b>	Preliminary Geotechnical Assessment : 2-30 Tempus Street, Rouse Hill, NSW
<b>Client</b>	FreeCity Rouse Hill Development Pty Ltd
<b>Document</b>	P2410429JR03V02
<b>Director</b>	Dr. Daniel Martens
<b>Manager</b>	Gray Taylor
<b>Principal Author</b>	Matthew Boulos

## Document History

Issue	Issue Date	Status	Description / Comment	Author	Reviewer	Approved
1.	2/12/2024	Draft	For Client Review	MB	WB	GT
2.	20/02/2025	Final	For Submission	MB	WB	GT
3	13/03/2025	Final	Updated SEARs	MB	WB	GT

© Copyright Martens & Associates Pty Ltd  
Suite 201, 20 George St, Hornsby, NSW 2077, Australia  
ACN 070 240 890 ABN 85 070 240 890  
P +61-2-9476-9999 | mail@martens.com.au | www.martens.com.au

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## Executive Summary

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This Preliminary Geotechnical Assessment has been prepared by Martens and Associates Ltd Pty to accompany a detailed State Significant Development Application (SSDA) for the mixed use development at 2-30 Tempus Street, Rouse Hill. The site is made up of one lot, being Lot 19 in DP 280013.

This report has been prepared to address the Secretary's Environmental Assessment Requirements (SEARs) issued for the project (SSD-76190964).

Key findings of this report are as follows:

- Intrusive investigation revealed the following generalised subsurface units:
  - Unit A: Fill comprising of inferred poorly compacted silty clay up to approximately 0.4 mbgl.
  - Unit B: Weathered rock comprising:
    - Unit B1: Highly weathered, inferred very low to low strength shale up to approximately 5.8 mbgl.
    - Unit B2: Distinctly weathered, inferred low to medium strength shale / laminate from depths between approximately 5.8 mbgl (BH102) and 7.9 mbgl (BH101 to investigation termination depth of 9.9 mbgl).
- Proposed bulk basement excavations up to approximately 40.584 mAHD is unlikely to intercept permanent groundwater.
- Weathered shale underlying the site can generally be categorised as non-saline. We consider that saline soil management strategies and preparation of a saline soil management plan are not required, subject to further laboratory testing of existing fill and any residual soil overlying shale bedrock during construction certificate stage prior to construction.

This report concludes that the proposed mixed use development is suitable and warrants approval subject to the implementation of the following mitigation measures.

- Excavations must be temporarily and permanently battered back / supported / retained to maintain excavation stability and limit potential adverse impacts on surrounding structures / neighbouring properties.
- Dilapidation surveys of adjacent structures may need to be carried out prior to excavation and following completion of construction to clearly identify damage caused by the excavation process.
- Shallow footings, such as pad and strip footings may be adopted as support for the new basement development, founding in very low to low strength shale. Considering expected column load from the proposed development, pile

foundations are likely to be required. Piles founding in at least very low to low strength shale, may be considered. Additional socket length may be required to achieve required uplift and lateral stability.

- The site is classified as a Class "P" site in accordance with AS 2870 (2011). A reclassification to "A" may be adopted subject to all shallow footings founding in shale bedrock.
- We recommend the following additional geotechnical investigation and assessment works are carried out at the construction certificate (CC) stage of work to develop the final design prior to construction:
  - Following site clearing and removal of fill mounds, undertake 3 – 4 additional borehole investigation with rock coring to better define the ground / rock conditions across the site and associated provided recommendations with respect to excavation support and foundation bearing capacity.
  - Review of final structural design by a senior geotechnical engineer to confirm adequate consideration of the geotechnical risks and adoption of the recommendations provided in this report.
  - Two to three additional groundwater monitoring wells be installed at strategic locations across the site to improve spatial and hydrogeological coverage. Based on a two level basement, at least one investigation hole is to be drilled to 15 m below ground level.
  - Monitoring of groundwater levels and quality be conducted at all wells over a minimum of three months to capture temporal variations.

Following the implementation of the above mitigation measures, the remaining impacts are considered appropriate.

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# 1 Introduction

## 1.1 Overview

Martens and Associates (**MA**) was engaged by Freecity Rouse Hill Development Pty Ltd to carry out a preliminary geotechnical assessment in relation to the proposed development at 2-30 Tempus Street, Rouse Hill, NSW (**the Site**). The site location is shown on Map 01, Attachment A.

This preliminary geotechnical assessment was undertaken in general accordance with the agreed scope of work outlined in MA's proposal P2410429BC01V01 dated 24 July, 2024.

## 1.2 SEARs

The purpose of the project is to facilitate the delivery of high-quality, diverse housing and commercial floor space at a strategically located site. The proposal seeks to deliver a built form outcome that responds appropriately to its location at the edge of Rouse Hill Town Centre and adjacent to Rouse Hill Metro Station and that is consistent with the desired future character of Rouse Hill.

This report has been prepared in response to the requirements contained within the Secretary's Environmental Assessment Requirements (SEARs) dated 30th September 2024 and issued for SSD-76190964. Specifically, this report has been prepared to respond to the SEARs requirement issued below.

**Table 1:** Contamination SEARs Requirements.

Item	Description of Requirement
13. Ground and Water Conditions	<ul style="list-style-type: none"> <li>• Assess potential impacts on soil resources and related infrastructure and riparian lands on and near the Site, including soil erosion, salinity and acid sulfate soils.</li> <li>• Provide a Surface and Groundwater Impact Assessment that assess potential impacts on:               <ul style="list-style-type: none"> <li>○ Surface water resources (quality and quantity) including related infrastructure, hydrology, dependent ecosystems, drainage lines, downstream assets and water courses.</li> <li>○ Groundwater resources in accordance with the <i>Groundwater Guidelines</i>.</li> </ul> </li> </ul>

It should be noted that this report only assesses impacts on soil resources, including soil erosion, salinity and acid sulfate soils (where applicable) and does not make an assessment on surface water resources.

The Preliminary Hydrogeological Assessment (MA, 2025), previously conducted by MA for the site, provides detailed information regarding potential groundwater impacts. Where necessary and relevant, the findings of this assessment have been referenced.



### 1.3 Proposed Development

The application seeks development consent for the development of an 11, 18 and 23 storey mixed use development at 2-30 Tempus Street, Rouse Hill. Specifically, the SSDA seeks development consent for:

- Site preparation works including removal of temporary planting, bulk excavation and earthworks.
- Construction and operation of an 11, 18 and 23 storey mixed use development, comprising:
  - Consolidated podium comprising ground level lobby, retail and wellness tenancies, and two levels of commercial floor space above
  - 216 co-living units within the 11-storey tower
  - 332 build-to-rent units across the 18 and 23-storey towers, including 227 dual key units
  - Rooftop internal and external amenity spaces on each tower to service the build-to-rent and co-living residents
- Landscaping and public domain works, including:
  - Retaining existing street trees
  - Provision of a deep soil landscaped buffer zone along the rear boundary
  - On-structure landscaping on each rooftop.
- Construction and use of two basement levels, accessed from White Hart Drive. We understand from architectural plans (Architectus 2025) that basement will have a finished floor level (**FFL**) of 40.584 mAHD to accommodate:
  - Approximately 111 car spaces
  - Motorcycle and bicycle parking
  - Loading dock facilities
- Extension and augmentation of services and infrastructure as required.
- Excavations will likely extend to all site boundaries. Therefore, excavation will extend into the zone of influence of neighbouring properties / other infrastructure to all site boundaries.

### 1.4 Assessment Purpose

The purpose of this preliminary geotechnical assessment is to support a State Significant Development Application (SSDA) and to provide a preliminary structural design advice and recommendations relating to the proposed development.

## 2 Investigation Scope and Laboratory Testing

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### 2.1 Investigation Scope of Work

#### 2.1.1 Field Investigation

Field investigations were conducted on 19 September 2024 and included:

1. Review of BYDA survey plans and locating of buried services on site.
2. A site walkover to review local geology, soil exposures, surface hydrology, topography, drainage and relevant site features.
3. Drilling of two boreholes (BH101 and BH102) using Martens' 4WD truck-mounted hydraulic drill rig up to 9.9 mbgl.
4. Four Dynamic Cone Penetrometer (DCP) tests (DCP101 to DCP104) up to DCP refusal depth of 0.5 mbgl.
5. One Standard Penetration Test (SPT) at 1.0 mbgl in BH101.
6. Installation of one groundwater monitoring well (MW) in BH101 (MW01).
7. Collection of soil and weathered rock samples from boreholes for laboratory testing and for future reference.

Investigation locations are shown on map GE03, Attachment A.

### 2.2 Laboratory Testing

Envirolab Services, a National Association of Testing Authorities (NATA) accredited laboratory carried out soil aggressivity testing on two soil samples.

Laboratory test certificate is provided in Appendix D.

### 3 Site Details and Subsurface Conditions

#### 3.1 Overview

The site is located on the southern edge of Rouse Hill Town Centre and to the east of Rouse Hill Metro Station.

To the east of the site across White Hart Drive is a large residential area comprising single dwellings and town houses.

To the south of the site across White Hart Drive is new residential flat development of approximately 6 to 12 storeys.

Open spaces are located in proximity to the site including Castlebrook Memorial Park to the south-west of the site across Windsor Road, Caddies Creek Park and Reserve to the south of the site and Iron Bark Ridge Reserve to the west of the site at Caddies Creek.

The site is identified as a ‘sleeve’ site in the Rouse Hill Town Centre Precinct Plan approval (DA 1581/2005/HB) where the intent is for future development to screen the existing big box retail and car parking structures behind. As the retail and car parking structures have already been constructed and are in operation, the site was temporarily treated with earth bents, landscaping and tree planting until the site is developed.

No other structures exist on the site.

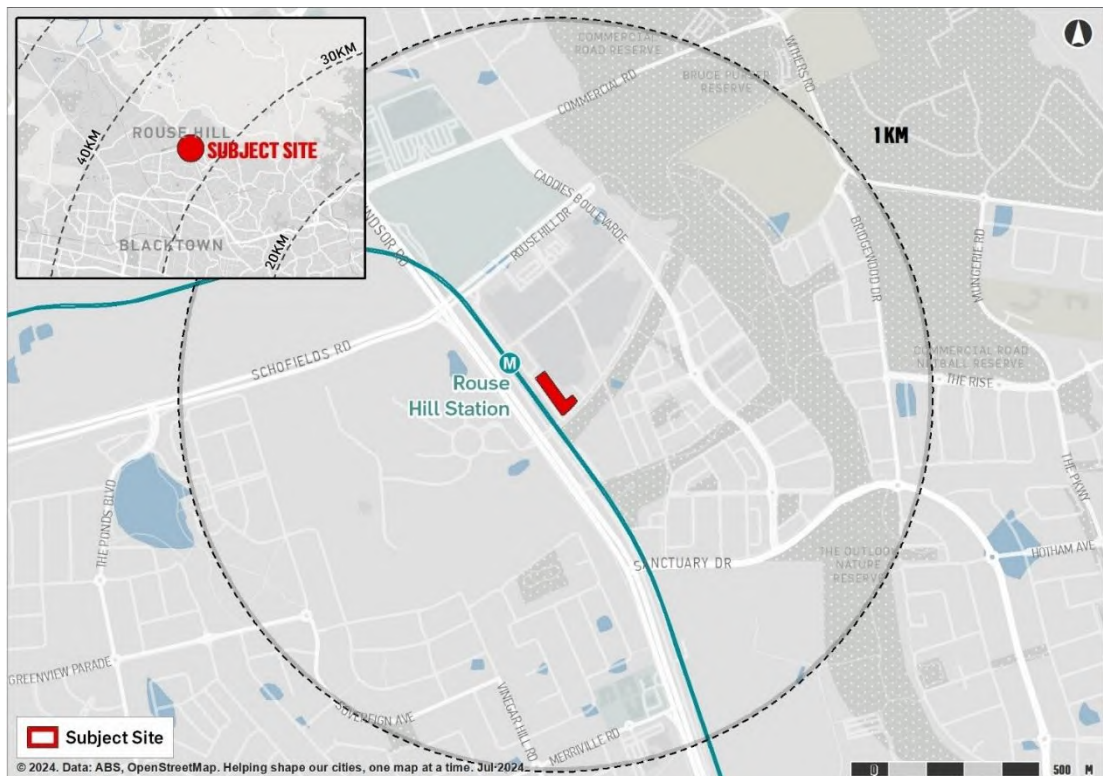


Figure 1: Regional Context. (Source: Urbis).

## 3.2 General Site Details

General Site details are summarised in Table 2.

**Table 2:** Summary of general Site details based on desktop reviews, and site walkover and field investigations.

Item	Description / Comment
Property address	2-30 Tempus Street, Rouse Hill, NSW ( <b>the site</b> ).
Lot / DP	Lot 19 in DP280013
LGA	The Hills Shire Council ( <b>'Council'</b> )
Site area	4385 m <sup>2</sup> (LTS, 2024)
Topography	The site is located within slightly undulating terrain, on the mid slope of an east / northeast facing slope. An inferred fill mound located within the central portion of the site.
Typical slopes, aspect, elevation	<p>The site has a west / south western aspect with overall grades less than approximately 10 %. The fill mound has steeper slopes of &lt;20 % towards the north east / east.</p> <p>Site elevation ranges between approximately 46.52 mAHD in the eastern portion of the site and 50.41 mAHD in the north western portion of the site (LTS, 2024). Refer to map GE01, Appendix A, for site topography.</p>
Expected geology	Ashfield Shale comprising drak-grey to black claystone-siltstone and fine sandstone-siltstone laminite (Clark and Jones, 1991). The site geology is shown on map GE02, Appendix A.
Expected landscape soil	The NSW Office of Environment and Heritage's (OEH) information system (eSPADE) indicates the site to be located in the Blacktown (bt) soil landscape, consisting of gently undulating rises on Wianamatta Group shales. This soil landscape is characterised by > 200 cm of soil on lower side slopes. This soil landscape is often associated with localised seasonal waterlogging, localised water erosion hazard, moderately reactive highly plastic subsoil and localised surface movement potential.
Acid Sulfate Soils (ASS)	The site isn't mapped as being impacted by ASS Class 1 to Class 5 and therefore an ASS assessment isn't necessary.
Vegetation	Shrubs and scattered young and mature trees.
Existing development	The site is currently undeveloped.
Neighbouring environment	<p>The site is bordered by:</p> <ul style="list-style-type: none"> <li>○ Market Lane and an at grade carpark to the northwest.</li> <li>○ An underground carpark and commercial building with a driveway from White Hart Drive to the northeast and east.</li> <li>○ White Hart Drive to the east / southeast.</li> <li>○ Tempus Street to the west / southwest.</li> </ul>
Drainage	Via overland flow towards the east / south east into Council stormwater drainage network along Tempus Street and White Hart Drive.

### 3.3 Subsurface Conditions

Investigation revealed the following generalised sub surface units underlie the investigation area:

Unit A: Fill comprising inferred poorly compacted silty clay encountered up to approximately 0.4 mbgl. Fill is inferred to have been placed under uncontrolled conditions during former development in surrounding areas.

Unit B: Weathered rock comprising:

Unit B1: Highly weathered, inferred very low to low strength shale, encountered up to between approximately 5.8 mbgl (BH102) and 7.9 mbgl (BH101).

Unit B2: Distinctly weathered, inferred low to medium strength shale / laminite, encountered from depths of between approximately 5.8 mbgl (BH102) and 7.9 mbgl (BH101) to investigation termination depth of 9.9 mbgl.

Encountered conditions are described in more detail in the borehole logs in Appendix B and associated explanatory notes in Appendix F. For DCP test results refer to Appendix C.

### 3.4 Groundwater

#### 3.4.1 NSW Department of Primary Industries Bore Search

A review of the NSW Department of Primary Industries Water (DPIW) groundwater bore database search indicated no groundwater bores with available groundwater data located within 500 m of the site.

#### 3.4.2 Borehole Observations

Groundwater inflow was not encountered during drilling of boreholes up to 9.9 mbgl.

#### 3.4.3 Groundwater Monitoring Wells

One groundwater monitoring well (MW) was installed in BH101 (MW01). A summary of the construction details of the monitoring well is presented in Table 3. Monitoring well log is provided in Appendix B.

**Table 3:** Summary of monitoring well construction details.

Location	Screened Material	Surface Level (mAHD) <sup>1</sup>	Depth of Top of Screen (mbgl/mAHD)	Depth of Bottom of Screen (mbgl/mAHD)	Slotted Screen Length (m)
MW01	Shale	48.5	1.9 / 46.6	9.9 / 38.6	8.0

**Notes:**

1. Surface level estimated from LTS (2024).

Data loggers (pressure transducers) were installed within MW01 on 19 September 2024 to monitor groundwater levels at a 15 minute frequency. Results of the groundwater monitoring will be presented in the Hydrogeological Impact Assessment Report (MA, 2025), following three months of monitoring.

### 3.4.4 Groundwater Level Measurements

A summary of dipped standing groundwater level readings in MW01 recorded on 19 September 2024 is provided in Table 4.

**Table 4:** Summary of standing groundwater levels measured in monitoring wells on 19/09/2024.

Location	Surface Level (mAHD) <sup>1</sup>	Standing Water Level (mbgl/mAHD)	
		19/09/2024	1/11/2024
MW01 (BH101)	48.50	9.70 / 38.80	9.72 / 38.78

**Notes:**

1. Surface level estimated from LTS (2024).

### 3.4.5 Groundwater Monitoring

Groundwater investigation findings and conclusions, as detailed in MA 2025, should be reviewed in conjunction with this report. Groundwater monitoring was conducted from 19 September 2024 to 24 December 2024, with results summarized in Table 5.

**Table 5:** Monitoring data summary (19/09/2024 – 24/12/2024).

Monitoring well	Surface level (mAHD)	Groundwater level (mbgl / mAHD)			Range (m)
		Minimum	Mean	Maximum	
MW01	48.5	9.45 / 39.05	9.38 / 39.12	9.26 / 39.24	0.18

### 3.4.6 Conclusions

Based on our observations during borehole drilling, dipped results and our engineering judgement, we conclude the following:

- Groundwater monitoring data indicated that groundwater levels at the Site is relatively consistent with groundwater levels between 39.05 mAHD and 39.24 mAHD.
- Based on measured groundwater levels during the preliminary monitoring period (39.05 mAHD – 39.24 mAHD) groundwater is not expected to be intercepted during the proposed development construction.
- Ephemeral perched seepage may be present in soil profile and / or at the soil / rock interface following prolonged or heavy rainfall events.

## 4 Salinity Assessment

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### 4.1 Documented Salinity Risk Potential

The 1:100,000 Salinity Potential in Western Sydney Map (DIPNR, 2002) indicates the site to be located in an area of moderate salinity potential.

### 4.2 Broad Scale Salinity Process

In producing the Salinity Potential Map, the Western Sydney Regional Organisation of Councils (WSROC) developed a number of alternative models of processes by which salinity may occur in Western Sydney (WSROC, 2003, pgs. 16 to 20).

A list of key broad scale salinity processes likely to impact the site, including summarised descriptions of each process, is presented in Table 6.

### 4.3 Signs of Potential Saline Soils at the Site

No obvious signs of saline conditions were observed across the site:

- Vegetation growth appeared healthy and uninhibited.
- No water marks or salt crystals were observed on the ground surface.
- Site surface drainage appeared good.
- No evidence of concentrated surface erosion was observed

### 4.4 Assessed Salinity Risk Potential

In Table 6, the broad scale salinity processes have been assessed in terms of likelihood of occurring at the site, considering the proposed development, site observations and investigation findings.

**Table 6:** Potential for broad scale salinity processes at the site.

Key Salinity Process	Description	Potential at subject site
Localised concentration of salinity	<p>Localised concentration of salts due to relatively high evaporation rates.</p> <p>Usually associated with waterlogged soil and poor drainage.</p> <p>Exacerbated by increased water use and / or blocking of surface and subsurface water flow associated with urban development.</p>	<p>Low – No evidence of localised salt concentration. No waterlogged soils or poor drainage observed. Former and existing surface drainage channels may have influenced site salinity.</p>
Shale soil landscapes	<p>In poorly drained duplex (texture contrast) soils, shallow subsurface water flows laterally across a clayey upper B-Horizon with salt usually accumulating in the clayey subsoil.</p> <p>Salt concentrations may increase where subsurface water accumulates and evaporates, e.g. on lower slopes or natural and constructed flats in mid-slope.</p> <p>Exacerbated by subsoils exposure through deep cutting, by installing buildings into the B-horizon and by impeding subsurface water flows.</p> <p>Highly dispersive, erodible and poorly draining sodic soils due to salinity.</p>	<p>Low to Moderate – The site is underlain by semi-impermeable clay fill, overlying shale.</p> <p>Minor evidence of impeded surface vegetation growth observed. No evidence of surface soil erosion observed.</p> <p>Bulk excavation into shale is proposed for the development.</p>
Deep groundwater salinity	<p>Brackish or saline groundwater rises to a level where, through capillary action in the soil, the water with dissolved salts reaches the ground surface and evaporates, resulting in localised salt concentration.</p> <p>Groundwater rises are typically caused by increased water infiltration, e.g. above average rainfall, vegetation loss, irrigation, increased water use in urban areas, construction of surface pits.</p> <p>Exacerbated by buildings or infrastructure intercepting the zone of groundwater level fluctuation.</p>	<p>Low – Groundwater level was not encountered during drilling of BH101 and BH102. The proposed development is not expected to intercept or raise groundwater levels.</p> <p>Proposed structures are to be constructed with appropriate drainage measures installed.</p>
Deeply weathered soil landscape	<p>High salt loads with high sulphate levels related to un-mapped deeply weathered soil landscapes beneath fluvial gravel, sand and clay.</p> <p>Usually in mid-slope or on hilltops affected by perched saline groundwater.</p>	<p>Low to Moderate – Encountered soils at the site are considered fill with depth to weathered rock approximately 0.4 mbgl. Limited residual soil overlying rock is expected at the site.</p>



## 4.5 Laboratory Test Results

### 4.5.1 Salinity Classification

Laboratory test results for salinity classification are summarised in Table 7.

**Table 7:** Salinity test results.

Sample ID <sup>1</sup>	Material	EC <sub>(1.5)</sub> (ds/m)	EC <sub>e</sub> (ds/m) <sup>2</sup>	Salinity Classification <sup>3</sup>
BH101 / 2.5 – 2.6	Weathered Shale	86	0.52	Non-Saline
BH101 / 5.7 – 5.8	Weathered Shale	150	0.90	Non-Saline

**Notes:**

1. Borehole#/Depth (mbgl).
2. Based on EC to EC<sub>e</sub> multiplication factors from Table 6.1 in Site investigation for Urban Salinity (2002) guidelines. A multiplication factor of 6 was adopted for Weathered Shale.
3. Based on Table 6.2 of DLWC (2002) where EC<sub>e</sub> <2 dS/m = non-saline, EC<sub>e</sub> 2-4 dS/m = slightly saline, EC<sub>e</sub> 4-8 dS/m = moderately saline, EC<sub>e</sub> 8-16 dS/m = very saline and EC<sub>e</sub> >16 dS/m = highly saline.

Limited laboratory test results indicate weathered shale underlying the site can generally be categorised non-saline. We consider that saline soil management strategies and preparation of a saline soil management plan are considered not required, subject to further laboratory testing of existing fill and any residual soil overlying shale bedrock during construction certificate stage prior to construction.

## 5 Geotechnical Assessment

### 5.1 Soil Aggressivity Testing

Laboratory test results for soil aggressivity are summarised in Table 8. Laboratory test certificate is provided in Appendix D.

**Table 8:** Summary of soil aggressivity results.

Sample ID <sup>1</sup>	Material	EC <sub>e</sub> (ds/m) <sup>2</sup>	Resistivity (ohm.cm) <sup>3</sup>	pH	Chloride (Cl) (mg/kg)	Sulphate (SO <sub>4</sub> ) (mg/kg)	Exposure Classification		
							AS 2159 <sup>4</sup>	AS 2159 <sup>5</sup>	AS 3600 <sup>6</sup>
BH101 / 2.5 – 2.6	Weathered Shale	0.52	1930	5.4	34	140	Mild	Mild	A2
BH101 / 5.7 – 5.8	Weathered Shale	0.90	1110	6.7	120	98	Non-aggressive	Mild	A1

**Notes:**

1. Borehole#/Depth (mbgl).
2. Based on EC to EC<sub>e</sub> multiplication factors from Table 6.1 in Site investigation for Urban Salinity (2002) guidelines. A multiplication factor of 6 was adopted for Weathered Shale.
3. Resistivity in soil is estimated using  $1/EC_e$  (dS/m)  $\times 10^3$ .
4. Exposure classification for concrete piles in soil based on Table 6.4.2(C) of AS 2159 (2009).
5. Exposure classification for steel piles in soil based on Table 6.5.2(C) of AS 2159 (2009).
6. Exposure classification for buried reinforced concrete based on Tables 4.8.1 and 4.8.2 of AS 3600 (2018).

In accordance with AS 2159 (2009), an exposure classification of 'Mild' should be adopted for buried concrete and steel piles. In accordance with AS3600 (2018), an exposure classification of 'A2' should be adopted for shallow reinforced concrete footings.

## 5.2 Preliminary Soil Properties

Preliminary soil and rock properties inferred from observations during borehole drilling, such as auger penetration resistance, SPT / DCP test results as well as engineering assumptions are summarised in Table 9.

**Table 9:** Preliminary soil properties.

Layer <sup>1</sup>	$\gamma_{in-situ}$ <sup>2</sup> (kN/m <sup>3</sup> )	$C_u$ <sup>3</sup> (kPa)	$C'$ <sup>4</sup> (kPa)	$\phi'$ <sup>5</sup> (deg)	$E'$ <sup>6</sup> (MPa)
FILL: Silty CLAY (poorly compacted)	16	25	2	24	4
WEATHERED ROCK: SHALE (highly weathered, inferred very low to low strength)	22	NA <sup>7</sup>	30	28	75
WEATHERED ROCK: SHALE (distinctly weathered, inferred low to medium strength)	23	NA <sup>7</sup>	60	30	150

**Notes:**

1. Refer to borehole logs in Appendix for material description details.
2. Average *in-situ* unit weight estimate.
3. Average undrained shear strength estimate, assuming normally consolidated clay.
4. Average drained (effective) cohesion estimate.
5. Average effective internal friction angle estimate, assuming drained conditions; may be dependent on rock defect conditions.
6. Average effective elastic modulus estimate.
7. Not applicable.

## 5.3 Risk of Slope Instability

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

## 6 Geotechnical Recommendations

---

### 6.1 Overview

Geotechnical recommendations for the proposed development are provided below. Further general geotechnical recommendations are provided in Appendix E.

### 6.2 Excavation and Vibration

Proposed basement excavations is likely to encounter fill or residual soils over very low to low strength and low to medium strength shale bedrock. Considering the ground conditions, the following excavation plant may be required:

- Soils: This should be readily excavated using conventional earthmoving equipment.
- Very low to low strength rock: This should generally be readily excavated with a 'toothed' bucket or a ripping tyne (or similar) although progress may be slower when containing low or higher strength bands.

If medium or higher strength rock encountered and to be excavated using a rock hammer, vibration management will be required in accordance with AS2187.2, Appendix J.

### 6.3 Excavation Support

Excavations must be temporarily and permanently battered back / supported / retained to maintain excavation stability and limit potential adverse impacts on surrounding structures / neighbouring properties. Appropriate support methodologies should be adopted by the excavation contractor and design engineer and approved by a geotechnical engineer.

If there is sufficient room to remain outside the zone of influence of surrounding structures / neighbouring properties, excavations in soils and weathered rock may be temporarily battered back at:

- 1V:2H for uncontrolled fill.
- 1V:1.5H for very low to low strength rock.
- 1V:1H for low to medium strength rock.

It is assumed that the temporary excavation batters will remain unsupported for no more than two months. Permanent batters, if adopted, should not exceed a grade of 1V:3H. Recommended batters are subject to inspection and approval by an experienced geotechnical engineer on site and should be followed by construction of permanent retaining structures.

Temporary shoring may include cantilevered / anchored concrete soldier pile walls with shotcrete infill panels. For shoring inside the zone of influence of adjoining existing structures / properties, and where excavation exceeds 3.5 m, closer pile spacing or

contiguous piles will be required. Soil and rock anchors, if adopted, must not be installed across site boundaries unless written confirmation of acceptance is obtained from neighbouring property / asset owners. Where installation of anchor is not possible, the soldier pile wall may be internally braced.

Temporary walls may be designed to provide long term retention with lateral restraint provided by basement and ground floor slabs.

#### **6.4 Earth Pressure Coefficients**

Preliminary shoring or retaining wall design, may adopt preliminary active, at rest and passive earth pressure coefficients, respectively, of:

- 0.42, 0.59, 2.37 for existing fill.
- 0.33, 0.50, 3.00 for very low strength rock.

#### **6.5 Dilapidation Surveys**

Dilapidation surveys of adjacent structures may need to be carried out prior to excavation and following completion of construction to clearly identify damage caused by the excavation process.

#### **6.6 Footing Systems and Safe Bearing Pressures**

Proposed excavation will likely expose very low to low strength shale at the basement excavation level across the excavation footprint. Shallow footings, such as pad and strip footings may be adopted as support for the new basement development, founding in very low to low strength shale. Considering expected column load from the proposed development, pile foundations are likely to be required. Piles founding in at least very low to low strength shale, may be considered. Additional socket length may be required to achieve required uplift and lateral stability.

Preliminary geotechnical parameters for design of shallow footings and piles are presented in Section 6.7, Table 10.

Footings should be designed by a suitably qualified and experienced structural or geotechnical engineer. All foundation excavations should be inspected by an experienced geotechnical engineer to confirm encountered conditions satisfy design assumptions.

#### **6.7 Preliminary Design Parameters**

Table 10 presents preliminary design parameters that may be adopted for design shallow and deep foundations.

**Table 10:** Preliminary foundation design parameters.

Layer	Shallow Footings	Piles <sup>1</sup>	
	ABC <sup>2</sup>	ABC <sup>2</sup>	ASF <sup>3</sup>
FILL: Silty CLAY (poorly compacted)	NA <sup>4</sup>	NA <sup>4</sup>	NA <sup>4</sup>
WEATHERED ROCK: SHALE (highly weathered, inferred very low to low strength)	350	700	50
WEATHERED ROCK: SHALE (distinctly weathered, inferred low to medium strength)	500	1000	100

**Notes:**

1. Assuming bored cast in-situ pile.
2. Allowable end bearing capacity (kPa) for shallow footings embedded at least 0.3 m and piles embedded at least 1.0 m or 1 × pile diameter (whichever is greater) into at least very low to low strength shale, subject to confirmation on site by a geotechnical engineer to confirm foundation conditions.
3. Allowable skin friction (kPa) for bored pile in compression, assuming intimate contact between pile and adjacent in-situ soils.
4. Not applicable.

## 6.8 Groundwater / Drainage Requirements

Proposed excavation is unlikely to intercept the permanent groundwater, subject to continuous groundwater monitoring results. Minor perched groundwater seepage inflow from excavation faces, during excavation, if encountered, should be managed by managed by sump and pump methods.

Appropriate drainage measures should be provided behind the shoring / retaining walls (e.g. strip drains) and beneath floor slabs to collect and divert perched groundwater water away from structures. Collected water should be discharge into council approved discharge points.

## 6.9 Soil Erosion Control

Removal of soil overburden should be performed in a manner that reduces the risk of sedimentation occurring in the Council stormwater system and on neighbouring lands. 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.

## 6.10 Site Classification

The site is classified as a Class "P" site in accordance with AS 2870 (2011). A reclassification to "A" may be adopted subject to all shallow footings founding in shale bedrock.

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

## 7 Proposed Additional Works

### 7.1 Further Works

We recommend the following additional geotechnical investigation and assessment works are carried out at the construction certificate (CC) stage of work to develop the final design prior to construction:

- Following site clearing and removal of fill mounds, undertake 3 – 4 additional borehole investigation with rock coring to better define the ground / rock conditions across the site and associated provided recommendations with respect to excavation support and foundation bearing capacity.
- Review of final structural design by a senior geotechnical engineer to confirm adequate consideration of the geotechnical risks and adoption of the recommendations provided in this report.
- Two to three additional groundwater monitoring wells be installed at strategic locations across the site to improve spatial and hydrogeological coverage. Based on a two level basement, at least one investigation hole is to be drilled to 15 m below ground level.
- Monitoring of groundwater levels and quality be conducted at all wells over a minimum of three months to capture temporal variations.

### 7.2 Construction Monitoring and Inspections

We recommend the following is inspected and monitored during construction of the project (Table 11).

**Table 11:** Recommended inspection / monitoring requirements during site works.

Scope of Works	Frequency/Duration	Who to Complete
Inspect excavation retention (shoring, retaining wall, anchors) installations and batter to assess need for additional support requirements.	Daily / As required <sup>2</sup>	Builder / MA <sup>1</sup>
Monitor groundwater seepage from excavation faces, if encountered, to assess stability of exposed materials.	When encountered	Builder / MA <sup>1</sup>
Monitor and analyse excavation-induced ground movement and settlement including retaining wall deflections.	Daily at on-set of excavation and as agreed thereafter	MA <sup>1</sup>
Inspect exposed material at foundation / subgrade level to verify suitability as foundation / lateral support / subgrade.	Prior to reinforcement set-up and concrete placement	MA <sup>1</sup>
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.



## 8 References

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- Architectus (2024) *Architectural Drawings*, Tempus Street, Rouse Hill, Project No. 240130, Drawing no. DA0091, DA0092, DA0200 – DA205, and DA0300 – DA0303, Revision P.00, dated 15 November 2024 (Architectus, 2024).
- Architectus (2025) *Architectural Drawings*, Tempus Street Rouse Hill, Drawing no. SK223, SK224, and SK225, Issue P.00, dated 10 February 2025 (Architectus, 2025).
- Clark N. R. and Jones D. C. (1991) *Penrith 1:100 000 Geological Sheet 9030*, 1st edition, Geological Survey of New South Wales, Sydney.
- Landcom (2004) *Managing Urban Stormwater: Soils and Construction*.
- Lilicrap A. and McGhie S. (2002) *Site Investigations for Urban Salinity*, Department of Land and Water Conservation (DLWC), Sydney, NSW, Australia.
- LTS Surveyors (2024) *Plan of Detail and Levels Over Lot 19 in DP 280013 at Rouse Hill Town Centre, Tempus Street, Rouse Hill*, Sheet no. 1, 4, 5 and 6, dated 19 September 2024 (LTS, 2024).
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- NSW Department of Environment & Heritage (2024), eSPADE, NSW soil and land information, [www.environment.nsw.gov.au](http://www.environment.nsw.gov.au), accessed 26.09.2024.
- 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 (2004) AS 1289.6.3.1:2004, Determination of the penetration resistance of a soil – Standard penetration test (SPT), SAI Global Limited.
- Standards Australia Limited (2017) AS 1726:2017, *Geotechnical site investigations*, SAI Global Limited.
- Standards Australia Limited (2009) AS 2159:2009, *Piling – Design and installation*, SAI Global Limited.
- Standards Australia Limited (2006) AS 2187.2:2006, *Explosives—Storage and use*, SAI Global Limited.
- Standards Australia Limited (2011) AS 2870:2011, *Residential slabs and footings*, SAI Global Limited.
- Standards Australia Limited (2018) AS 3600:2018, *Concrete Structures*, SAI Global Limited.
- Western Sydney Regional Organisation of Councils (WSROC, 2003) *Western Sydney Salinity Code of Practice*.

## 9 Appendix A – Map and Figure

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**Legend**

- Site Boundary
- Cadastre
- Countours (LTS, 2024)



1:1000 @ A3

Viewport

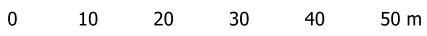
- Notes:**
- Aerial from Nearmaps (2024);
  - Cadastre and Site Boundary from NSW Clip & Ship (2024);
  - Contours from LTS Surveyors (2024).

Map Title / Figure:  
**Site Topography**

<b>GE01</b>	Map
2 Tempus Street, Rouse Hill, NSW	Site
Proposed Multi-Storey Mixed Use Development	Project
Preliminary Geotechnical Assessment	Sub-Project
Freecity Rouse Hill Development Pty Ltd	Client
13/03/2025	Date

**Legend**

- Site Boundary
- Cadastre
- Geology**
- Rwa - Ashfield Shale



1:1000 @ A3

Viewport

**Notes:**  
 - Aerial from Nearmaps (2024);  
 - Cadastre and Site Boundary from NSW Clip & Ship (2024);  
 - Geology from NSW Seamless Geology (2024).

Map Title / Figure:  
**Site Geology**

**Legend**

- Site Boundary
- Cadastre
- ⊕ Borehole and DCP Locations
- ⊕ Monitoring Well Location



0 10 20 30 40 50 m

1:1000 @ A3

Viewport

Notes:  
 - Aerial from Nearmaps (2024);  
 - Cadastre and Site Boundary from NSW Clip & Ship (2024).

Map Title / Figure:  
**Geotechnical Investigation Plan**

<b>GE03</b>	Map
2 Tempus Street, Rouse Hill, NSW	Site
Proposed Multi-Storey Mixed Use Development	Project
Preliminary Geotechnical Assessment	Sub-Project
Freecity Rouse Hill Development Pty Ltd	Client
13/03/2025	Date

## 10 Appendix B – Borehole Logs

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CLIENT	Freecity Rouse Hill Developments Pty Ltd	COMMENCED	19/09/2024	COMPLETED	19/09/2024	<b>REF BH101</b>	
PROJECT	Geotechnical Investigation	LOGGED	MB	CHECKED	WB	Sheet 1 OF 1	
SITE	2 Tempus St, Rouse Hill, NSW	GEOLOGY	Ashfield Shale	VEGETATION	NIL	PROJECT NO. P2410429	
EQUIPMENT	4WD truck-mounted hydraulic drill rig	LONGITUDE	150.92525	RL SURFACE	48.5 m	DATUM	AHD
EXCAVATION DIMENSIONS	ø100 mm x 9.90 m depth	LATITUDE	-33.692594	ASPECT	East	SLOPE	< 10%

Drilling			Sampling			Field Material Description							
METHOD	PENETRATION RESISTANCE	WATER	DEPTH (metres)	DEPTH RL	SAMPLE OR FIELD TEST	RECOVERED	GRAPHIC LOG	USCS / ASCS CLASSIFICATION	SOIL/ROCK MATERIAL DESCRIPTION	MOISTURE CONDITION	CONSISTENCY	DENSITY	STRUCTURE AND ADDITIONAL OBSERVATIONS
AD/V	L-M		48.50	0.40	0.2-0.3/S/1 D 0.20-0.30 m	█	⊗	CL	FILL: Silty CLAY; low plasticity; light brown, brown; with shale gravels; inferred poorly compacted.	M (<PL)			FILL
			48.10	0.80	0.5-0.6/S/1 D 0.50-0.60 m	█			SHALE; highly weathered; light brown; inferred very low strength.				WEATHERED ROCK 0.60: V-bit refusal.
			47.70	1.10	0.9-1.0/S/1 D 0.90-1.00 m	█			Grading to red brown.				
			47.40	1.50	SPT 1.00-1.15 m 14, HB N = refusal	█			Grading to grey.				
	L-M		47.00	1.60	1.6-1.7/S/1 D 1.60-1.70 m	█			Grading to red brown, larger shale fragments.				
			2	2.40	2.5-2.6/S/1 D 2.50-2.60 m	█			Grading to light brown, brown.				
				3.10					Inferred low strength				
			4	4.50									
	M	Not Encountered		44.00					Grading to dark grey, grey.				
					4.9-5.0/S/1 D 4.90-5.00 m	█							
				5.70	5.7-5.8/S/1 D 5.70-5.80 m	█			Grading to grey, pale grey, dark grey.				
			6	6.20					Grading to pale grey, grey; inferred low strength; water added.				
	M-H			42.30									
	H												
	M												
	H												
	M-H												
				7.90									
			8	40.60					SHALE; distinctly weathered; orange brown, light brown; inferred low to medium strength.				
					8.5-8.6/S/1 D 8.50-8.60 m	█							
	H												
				9.90									
			10						Hole Terminated at 9.90 m Refusal				

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

MARTENS 2.00.LIB.GLB Log MARTENS BOREHOLE P2410429BH101-102.GPJ <-DrawingFile>> 13/03/2025 11:48 10.02.00.04 D:\git\Lab and In Situ Tool - DGD [Lib: Martens 2.00 2016-11-13 Proj: Martens 2.00 2016-11-13]



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

**Engineering Log -  
BOREHOLE**

CLIENT	Freecity Rouse Hill Developments Pty Ltd	COMMENCED	19/09/2024	COMPLETED	19/09/2024	REF <b>BH101 / MW01</b> Sheet 1 OF 1 PROJECT NO. P2410429	
PROJECT	Geotechnical Investigation	LOGGED	MB	CHECKED	WB		
SITE	2 Tempus St, Rouse Hill, NSW	GEOLOGY	Ashfield Shale	VEGETATION	NIL		
EQUIPMENT	4WD truck-mounted hydraulic drill rig	LONGITUDE	150.92525	RL SURFACE	48.5 m	DATUM	AHD
EXCAVATION DIMENSIONS	9.90 m depth	LATITUDE	-33.692594	ASPECT	East	SLOPE	< 10%

Drilling			Sampling			Field Material Description						
METHOD	PENETRATION RESISTANCE	WATER	DEPTH (metres)	DEPTH RL	SAMPLE OR FIELD TEST	RECOVERED	GRAPHIC LOG	USCS / ASCS CLASSIFICATION	SOIL/ROCK MATERIAL DESCRIPTION	MOISTURE CONDITION	CONSISTENCY DENSITY	PIEZOMETER DETAILS
AD/1	L-M											ID MW01 Static Water Level
AD/1	L-M	Not Encountered	48.50	0.2-0.3/S/1 D	0.20-0.30 m	█	█	CL	FILL: Silty CLAY; low plasticity; light brown, brown; with shale gravels; inferred poorly compacted.	M (<PL)		
			0.40	0.20-0.30 m								
			48.10	0.5-0.6/S/1 D	0.50-0.60 m	█	█			SHALE; highly weathered; light brown; inferred very low strength.		
			0.80									
			47.70	0.9-1.0/S/1 D	0.90-1.00 m	█	█			Grading to red brown.		
			1.10	0.90-1.00 m								
			47.40	SPT 1.00-1.15 m	14, HB	█	█			Grading to grey.		
			1.50	N = refusal								
			47.00	1.6-1.7/S/1 D	1.60-1.70 m	█	█			Grading to red brown, larger shale fragments.		
			2.40									
46.10	2.5-2.6/S/1 D	2.50-2.60 m	█	█				Grading to light brown, brown.				
3.10												
45.40								Inferred low strength				
4.50												
44.00								Grading to dark grey, grey.				
5.70	4.9-5.0/S/1 D	4.90-5.00 m	█	█								
42.80	5.7-5.8/S/1 D	5.70-5.80 m	█	█				Grading to grey, pale grey, dark grey.				
6.20												
42.30								Grading to pale grey, grey; inferred low strength; water added.				
7.90												
40.60												
8.50	8.5-8.6/S/1 D	8.50-8.60 m	█	█				SHALE; distinctly weathered; orange brown, light brown; inferred low to medium strength.				
9.90												
									Hole Terminated at 9.90 m Refusal			

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

MARTENS 2.00.LIB.GLB Log MARTENS BOREHOLE P2410429BH101-102.GPJ <-DrawingFile>> 13/03/2025 11:48 10.02.00.04 D:\gel\lib and in situ tool - DGT [Lib: Martens 2.00 2016-11-13 Proj: Martens 2.00 2016-11-13]



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Phone: (02) 9476 9999 Fax: (02) 9476 8767  
mail@martens.com.au WEB: http://www.martens.com.au

**Engineering Log -  
TEST**





## 11 Appendix C – DCP Test Results

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## 12 Appendix D – Laboratory Test Certificates

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## CERTIFICATE OF ANALYSIS 362329

### Client Details

<b>Client</b>	Martens & Associates Pty Ltd
<b>Attention</b>	Matthew Boulos
<b>Address</b>	Suite 201, 20 George St, Hornsby, NSW, 2077

### Sample Details

<b>Your Reference</b>	<u>Geotechnical Investigation:2-30 Tempus Street</u>
<b>Number of Samples</b>	2 Soil
<b>Date samples received</b>	20/09/2024
<b>Date completed instructions received</b>	20/09/2024

### Analysis Details

Please refer to the following pages for results, methodology summary and quality control data.  
 Samples were analysed as received from the client. Results relate specifically to the samples as received.  
 Results are reported on a dry weight basis for solids and on an as received basis for other matrices.

### Report Details

<b>Date results requested by</b>	27/09/2024
<b>Date of Issue</b>	27/09/2024
NATA Accreditation Number 2901. This document shall not be reproduced except in full.	
Accredited for compliance with ISO/IEC 17025 - Testing. <b>Tests not covered by NATA are denoted with *</b>	

**Results Approved By**  
 Jenny He, Senior Chemist

**Authorised By**  
 Nancy Zhang, Laboratory Manager

Misc Inorg - Soil			
Our Reference		362329-1	362329-2
Your Reference	UNITS	10429/BH101/2.5 -2.6	10429/BH101/5.7 -5.8
Date Sampled		19/09/2024	19/09/2024
Type of sample		Soil	Soil
Date prepared	-	23/09/2024	23/09/2024
Date analysed	-	23/09/2024	23/09/2024
pH 1:5 soil:water	pH Units	5.4	6.7
Electrical Conductivity 1:5 soil:water	µS/cm	86	150
Chloride, Cl 1:5 soil:water	mg/kg	34	120
Sulphate, SO4 1:5 soil:water	mg/kg	140	98

**Client Reference: Geotechnical Investigation:2-30 Tempus Street**

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

**Client Reference: Geotechnical Investigation:2-30 Tempus Street**

QUALITY CONTROL: Misc Inorg - Soil				Duplicate				Spike Recovery %		
Test Description	Units	PQL	Method	Blank	#	Base	Dup.	RPD	LCS-1	[NT]
Date prepared	-			23/09/2024	[NT]	[NT]	[NT]	[NT]	23/09/2024	[NT]
Date analysed	-			23/09/2024	[NT]	[NT]	[NT]	[NT]	23/09/2024	[NT]
pH 1:5 soil:water	pH Units		Inorg-001	[NT]	[NT]	[NT]	[NT]	[NT]	99	[NT]
Electrical Conductivity 1:5 soil:water	µS/cm	1	Inorg-002	<1	[NT]	[NT]	[NT]	[NT]	100	[NT]
Chloride, Cl 1:5 soil:water	mg/kg	10	Inorg-081	<10	[NT]	[NT]	[NT]	[NT]	103	[NT]
Sulphate, SO4 1:5 soil:water	mg/kg	10	Inorg-081	<10	[NT]	[NT]	[NT]	[NT]	101	[NT]



## Result Definitions

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

### Quality Control Definitions

<b>Blank</b>	This is the component of the analytical signal which is not derived from the sample but from reagents, glassware etc, can be determined by processing solvents and reagents in exactly the same manner as for samples.
<b>Duplicate</b>	This is the complete duplicate analysis of a sample from the process batch. If possible, the sample selected should be one where the analyte concentration is easily measurable.
<b>Matrix Spike</b>	A portion of the sample is spiked with a known concentration of target analyte. The purpose of the matrix spike is to monitor the performance of the analytical method used and to determine whether matrix interferences exist.
<b>LCS (Laboratory Control Sample)</b>	This comprises either a standard reference material or a control matrix (such as a blank sand or water) fortified with analytes representative of the analyte class. It is simply a check sample.
<b>Surrogate Spike</b>	Surrogates are known additions to each sample, blank, matrix spike and LCS in a batch, of compounds which are similar to the analyte of interest, however are not expected to be found in real samples.
Australian Drinking Water Guidelines recommend that Thermotolerant Coliform, Faecal Enterococci, & E.Coli levels are less than 1cfu/100mL. The recommended maximums are taken from "Australian Drinking Water Guidelines", published by NHMRC & ARMC 2011.	
The recommended maximums for analytes in urine are taken from "2018 TLVs and BEIs", as published by ACGIH (where available). Limit provided for Nickel is a precautionary guideline as per Position Paper prepared by AIOH Exposure Standards Committee, 2016.	
Guideline limits for Rinse Water Quality reported as per analytical requirements and specifications of AS 4187, Amdt 2 2019, Table 7.2	

### Laboratory Acceptance Criteria

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

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

Spikes for Physical and Aggregate Tests are not applicable.

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

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

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

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

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

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

Where matrix spike recoveries fall below the lower limit of the acceptance criteria (e.g. for non-labile or standard Organics <60%), positive result(s) in the parent sample will subsequently have a higher than typical estimated uncertainty (MU estimates supplied on request) and in these circumstances the sample result is likely biased significantly low.

Measurement Uncertainty estimates are available for most tests upon request.

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

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

## 13 Appendix E – General Geotechnical Recommendations

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# Geotechnical Recommendations

## Important Recommendations About Your Site (1 of 2)

*These general geotechnical recommendations have been prepared by Martens to help you deliver a safe work site, to comply with your obligations, and to deliver your project. Not all are necessarily relevant to this report but are included as general reference. Any specific recommendations made in the report will override these recommendations.*

### **Batter Slopes**

Excavations in soil and extremely low to very low strength rock exceeding 0.75 m depth should be battered back at grades of no greater than 1 Vertical (V) : 2 Horizontal (H) for temporary slopes (unsupported for less than 1 month) and 1 V : 3 H for longer term unsupported slopes.

Vertical excavation may be carried out in medium or higher strength rock, where encountered, subject to inspection and confirmation by a geotechnical engineer. Long term and short term unsupported batters should be protected against erosion and rock weathering due to, for example, stormwater run-off.

Batter angles may need to be revised depending on the presence of bedding partings or adversely oriented joints in the exposed rock, and are subject to on-site inspection and confirmation by a geotechnical engineer. Unsupported excavations deeper than 1.0 m should be assessed by a geotechnical engineer for slope instability risk.

Any excavated rock faces should be inspected during construction by a geotechnical engineer to determine whether any additional support, such as rock bolts or shotcrete, is required.

### **Earthworks**

Earthworks should be carried out following removal of any unsuitable materials and in accordance with AS3798 (2007). A qualified geotechnical engineer should inspect the condition of prepared surfaces to assess suitability as foundation for future fill placement or load application.

Earthworks inspections and compliance testing should be carried out in accordance with Sections 5 and 8 of AS3798 (2007), with testing to be carried out by a National Association of Testing Authorities (NATA) accredited testing laboratory.

### **Excavations**

All excavation work should be completed with reference to the *Work Health and Safety (Excavation Work) Code of Practice (2015)*, by Safe Work Australia. Excavations into rock may be undertaken as follows:

1. Extremely low to low strength rock - conventional hydraulic earthmoving equipment.
2. Medium strength or stronger rock - hydraulic earthmoving equipment with rock hammer or ripping tyne attachment.

Exposed rock faces and loose boulders should be monitored to assess risk of block / boulder movement, particularly as a result of excavation vibrations.

### **Fill**

Subject to any specific recommendations provided in this report, any fill imported to site is to comprise approved material with maximum particle size of two thirds the final layer thickness. Fill should be placed in horizontal layers of not more than 300 mm loose thickness, however, the layer thickness should be appropriate for the adopted compaction plant.

### **Foundations**

All exposed foundations should be inspected by a geotechnical engineer prior to footing construction to confirm encountered conditions satisfy design assumptions and that the base of all excavations is free from loose or softened material and water. Water that has ponded in the base of excavations and any resultant softened material is to be removed prior to footing construction.

Footings should be constructed with minimal delay following excavation. If a delay in construction is anticipated, we recommend placing a concrete blinding layer of at least 50 mm thickness in shallow footings or mass concrete in piers / piles to protect exposed foundations.

A geotechnical engineer should confirm any design bearing capacity values, by further assessment during construction, as necessary.

### **Shoring - Anchors**

Where there is a requirement for either soil or rock anchors, or soil nailing, and these structures penetrate past a property boundary, appropriate permission from the adjoining land owner must be obtained prior to the installation of these structures.

### **Shoring - Permanent**

Permanent shoring techniques may be used as an alternative to temporary shoring. The design of such structures should be in accordance with the findings of this report and any further testing recommended by this report. Permanent shoring may include [but not be limited to] reinforced block work walls, contiguous and semi contiguous pile walls, secant pile walls and soldier pile walls with or without reinforced shotcrete infill panels. The choice of shoring system will depend on the type of structure, project budget and site specific geotechnical conditions.

Permanent shoring systems are to be engineer designed and backfilled with suitable granular

## Important Recommendations About Your Site (2 of 2)

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.

## 14 Appendix F – Notes about this Report

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# Information

## 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.



### Definitions

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

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

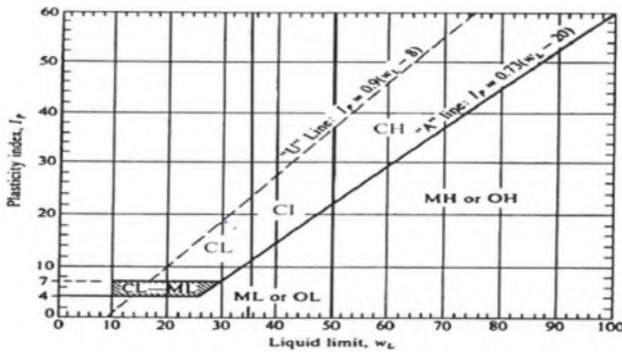
### Particle Size

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

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

### Plasticity Properties

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



### Soil Moisture Condition

#### Coarse Grained (Granular) Soil:

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

#### Fine Grained (Cohesive) Soil:

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

<sup>1</sup> Plastic Limit (PL): Moisture content at which soil becomes too dry to be in a plastic condition.

<sup>2</sup> Liquid Limit (LL): Moisture content at which soil passes from plastic to liquid state.

### Consistency of Cohesive Soils

Cohesive soils refer to predominantly clay materials.

(Note: consistency is affected by soil moisture condition at time of measurement)

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

### Density of Granular Soils

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

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

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








### Minor Components








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

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






### Symbols for Soils and Other

#### SOILS

	COBBLES/BOULDERS
	GRAVEL (GP or GW)
	Silty GRAVEL (GM)
	Clayey GRAVEL (GC)
	SAND (SP or SW)
	Silty SAND (SM)
	Clayey SAND (SC)

	SILT (ML or MH)
	ORGANIC SILT or CLAY (OH or OL)
	CLAY (CL, CI or CH)
	Silty CLAY
	Sandy CLAY
	PEAT (Pt)
	Gravelly CLAY

#### OTHER

	FILL
	TALUS
	ASPHALT
	CONCRETE
	TOPSOIL

### Unified Soil Classification Scheme (USCS)

FIELD IDENTIFICATION PROCEDURES (Excluding particles larger than 63 mm and basing fractions on estimated mass)					USCS	Primary Name	
COARSE GRAINED SOILS More than 65 % of material less than 63 mm is larger than 0.075 mm	(A 0.075 mm particle is about the smallest particle visible to the naked eye)	GRAVELS More than half of coarse fraction is larger than 2.36 mm.	GRAVEL and GRAVEL-SAND mixtures (±5% fines)	Wide range in grain size and substantial amounts of all intermediate particle sizes; not enough fines to bind coarse grains; no dry strength	GW	GRAVEL	
			GRAVEL-SILT and GRAVEL-SAND mixtures (±5% fines)	Predominantly one size or a range of sizes with some intermediate sizes missing; not enough fines to bind coarse grains; no dry strength	GP	GRAVEL	
			GRAVEL-SILT and GRAVEL-SAND mixtures (±12% fines) <sup>1</sup>	With excess non-plastic fines (for identification procedures see ML below); zero to medium dry strength; may also contain sand	GM	Silty GRAVEL	
			GRAVEL-SILT and GRAVEL-SAND mixtures (±12% fines) <sup>1</sup>	With excess plastic fines (for identification procedures see CL below); medium to high dry strength; may also contain sand	GC	Clayey GRAVEL	
		SANDS More than half of coarse fraction is smaller than 2.36 mm	SAND and GRAVEL-SAND mixtures (±5% fines)	Wide range in grain sizes and substantial amounts of all intermediate sizes; not enough fines to bind coarse grains; no dry strength.	SW	SAND	
			SAND-SILT and SAND-CLAY mixtures (±12% fines) <sup>1</sup>	Predominantly one size or a range of sizes with some intermediate sizes missing; not enough fines to bind coarse grains; no dry strength	SP	SAND	
			SAND-SILT and SAND-CLAY mixtures (±12% fines) <sup>1</sup>	With excess non-plastic fines (for identification procedures see ML below); zero to medium dry strength;	SM	Silty SAND	
			SAND-SILT and SAND-CLAY mixtures (±12% fines) <sup>1</sup>	With excess plastic fines (for identification procedures see CL below); medium to high dry strength	SC	Clayey SAND	
FINE GRAINED SOILS More than 35 % of material less than 63 mm is smaller than 0.075 mm	(A 0.075 mm particle is about the smallest particle visible to the naked eye)	IDENTIFICATION PROCEDURES ON FRACTIONS < 0.2 MM					
		DRY STRENGTH (Crushing Characteristics)	DILATANCY	TOUGHNESS	DESCRIPTION	USCS	Primary Name
		None to Low	Quick to Slow	Low	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands or silt with low plasticity <sup>2</sup>	ML	SILT <sup>3</sup>
		Medium to High	None to Slow	Medium	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays	CL (or CL <sup>4</sup> )	CLAY
		Low to Medium	Slow	Low	Organic silts and organic silty clays of low plasticity	OL	Organic SILT or CLAY
		Low to Medium	None to Slow	Low to Medium	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts	MH	SILT <sup>3</sup>
		High to Very High	None	High	Inorganic clays of high plasticity, fat clays	CH	CLAY
		Medium to High	None to Very Slow	Low to Medium	Organic clays of medium to high plasticity, organic silt of high plasticity	OH	Organic SILT or CLAY
HIGHLY ORGANIC SOILS	Readily identified by colour, odour, spongy feel and frequently by fibrous texture				Pt	PEAT	
<b>Notes:</b>							
1. Between 5% and 12% - dual classification, e.g. GP-GM.							
2. Low Plasticity Clay – Liquid Limit W <sub>L</sub> ≤35%; Medium Plasticity Clay – Liquid limit W <sub>L</sub> >35%, ≤50%; High Plasticity Clay - Liquid limit W <sub>L</sub> > 50%.							
3. Low Plasticity Silt – Liquid Limit W <sub>L</sub> ≤50%; High Plasticity Silt - Liquid limit W <sub>L</sub> > 50%.							
4. CI may be adopted for clay of medium plasticity to distinguish from clay of low plasticity.							

### Soil Agricultural Classification Scheme

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

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

### Symbols for Rock

#### SEDIMENTARY ROCK



BRECCIA



CONGLOMERATE



CONGLOMERATIC SANDSTONE



SANDSTONE/QUARTZITE



SILTSTONE



MUDSTONE/CLAYSTONE



SHALE



COAL



LIMESTONE



LITHIC TUFF

#### IGNEOUS ROCK



GRANITE



DOLERITE/BASALT

#### METAMORPHIC ROCK



SLATE, PHYLLITE, SCHIST



GNEISS



METASANDSTONE



METASILTSTONE



METAMUDSTONE

### Definitions

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

**Rock Material** The intact rock that is bounded by defects.

**Rock Defect** Discontinuity, fracture, break or void in the material or minerals across which there is little or no tensile strength.

**Rock Structure** The nature and configuration of the different defects within the rock mass and their relationship to each other.

**Rock Mass** The entirety of the system formed by all of the rock material and all of the defects that are present.

### Degree of Weathering

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

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

#### Notes:

1 RS and EW material is described using soil descriptive terms.

2. The term "Distinctly Weathered" (DW) may be used to cover the range of substance weathering between EW and SW

### Rock Strength

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

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

### Degree of Fracturing

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

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

### Rock Core Recovery

TCR = Total Core Recovery

SCR = Solid Core Recovery

RQD = Rock Quality Designation

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

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

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

### Rock Strength Tests

- ▼ Point load strength Index (Is50) - axial test (MPa)
- ▶ Point load strength Index (Is50) - diametral test (MPa)
- Uniaxial compressive strength (UCS) (MPa)

### Defect Type Abbreviations and Descriptions

Defect Type (with inclination given)	Planarity	Roughness
BP Bedding plane parting	PI Planar	Pol Polished
FL Foliation	Cu Curved	Sl Slickensided
CL Cleavage	Un Undulating	Sm Smooth
JT Joint	St Stepped	Ro Rough
FC Fracture	Ir Irregular	VR Very rough
SZ/SS Sheared zone/ seam (Fault)	Dis Discontinuous	
CZ/CS Crushed zone/ seam	<b>Thickness</b>	<b>Coating or Filling</b>
DZ/DS Decomposed zone/ seam	Zone > 100 mm	Cn Clean
FZ Fractured Zone	Seam > 2 mm < 100 mm	Sn Stain
IS Infilled seam	Plane < 2 mm	Ct Coating
VN Vein		Vnr Veneer
CO Contact		Fe Iron Oxide
HB Handling break		X Carbonaceous
DB Drilling break		Qz Quartzite
		MU Unidentified mineral
	<b>Inclination</b>	
	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.	

# Test, Drill and Excavation Methods

## Explanation of Terms (1 of 3)

### 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<sub>50</sub> (50 mm internal diameter thin walled tube), into soils and withdrawing a soil sample in a relatively undisturbed state. Such samples yield information on structure and strength and are necessary for laboratory determination of shear strength and compressibility. Undisturbed sampling is generally effective only in cohesive soils. Other sampling methods may be used. Details of the type and method of sampling are given in the report.

### Drilling / Excavation Methods

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

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

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

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

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

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

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

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

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

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

### In-situ Testing and Interpretation

#### Cone Penetrometer Testing (CPT)

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

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

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

- (i) Cone resistance ( $q_c$ ) - the actual end bearing force divided by the cross sectional area of the cone, expressed in MPa.
- (ii) Sleeve friction ( $q_f$ ) - the frictional force of the sleeve divided by the surface area, expressed in kPa.
- (iii) Friction ratio - the ratio of sleeve friction to cone resistance, expressed in percent.

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

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

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

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

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

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

# Test, Drill and Excavation Methods

## Explanation of Terms (2 of 3)

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

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

### Standard Penetration Testing (SPT)

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

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

- (i) Where full 450 mm penetration is obtained with successive blow counts for each 150 mm of say 4, 6 and 7 blows:  
as 4, 6, 7  
N = 13
- (ii) Where the test is discontinued, short of full penetration, say after 15 blows for the first 150mm and 30 blows for the next 40mm  
as 15, 30/40 mm.

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

### Dynamic Cone (Hand) Penetrometers

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

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

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

### Pocket Penetrometers

The pocket (hand) penetrometer (PP) is typically a light weight spring hand operated device with a stainless steel

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

$$q_u = 2 \times C_u.$$

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

### Test Pit / Borehole Logs

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

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

### Laboratory Testing

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

### Ground Water

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

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

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

# Test, Drill and Excavation Methods

## Explanation of Terms (3 of 3)

### DRILLING / EXCAVATION METHOD

HA	Hand Auger	RD	Rotary Blade or Drag Bit	NQ	Diamond Core - 47 mm
AD/V	Auger Drilling with V-bit	RT	Rotary Tricone bit	NMLC	Diamond Core – 51.9 mm
AD/T	Auger Drilling with TC-Bit	RAB	Rotary Air Blast	HQ	Diamond Core – 63.5 mm
AS	Auger Screwing	RC	Reverse Circulation	HMLC	Diamond Core – 63.5 mm
HSA	Hollow Stem Auger	CT	Cable Tool Rig	DT	Diatube Coring
S	Excavated by Hand Spade	PT	Push Tube	NDD	Non-destructive digging
BH	Tractor Mounted Backhoe	PC	Percussion	PQ	Diamond Core - 83 mm
JET	Jetting	E	Tracked Hydraulic Excavator	X	Existing Excavation

### SUPPORT

Nil	No support	S	Shotcrete	RB	Rock Bolt
C	Casing	Sh	Shoring	SN	Soil Nail
WB	Wash bore with Blade or Bailer	WR	Wash bore with Roller	T	Timbering

### WATER

- Water level at date shown  
 Water inflow  
 Partial water loss  
 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	CPT	Static cone penetration test
4,7,11	4,7,11 = Blows per 150mm.	CPTu	CPT with pore pressure (u) measurement
N=18	'N' = Recorded blows per 300mm penetration following 150mm seating	PP	Pocket penetrometer test expressed as instrument reading (kPa)
DCP	Dynamic Cone Penetration test to AS1289.6.3.2-1997.	FP	Field permeability test over section noted
	'n' = Recorded blows per 150mm penetration	VS	Field vane shear test expressed as uncorrected shear strength (sv = peak value, sr = residual value)
<b>Notes:</b>		PM	Pressuremeter test over section noted
RW	Penetration occurred under rod weight only	PID	Photoionisation Detector reading in ppm
HW	Penetration occurred under hammer and rod weight only	WPT	Water pressure tests
20/100mm	Where practical refusal or hammer double bouncing occurred, blows and penetration for that interval are reported (e.g. 20 blows for 100 mm penetration)		

### 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		