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Integrated Practical Solutions

Report on Geotechnical Investigation

Proposed Apartments Development
20 Helm Street, Mount Pleasant, WA

Prepared for
Pyramid Constructions (WA) Pty Ltd

Project 201795.00
March 2021

Douglas Partners
Geotechnics | Environment | Groundwater

Document History

Document details

Project No.	201795.00	Document No.	R.001.Rev0
Document title	Report on Geotechnical Investigation Proposed Apartments Development		
Site address	20 Helm Street, Mount Pleasant, WA		
Report prepared for	Pyramid Constructions (WA) Pty Ltd		
File name	201795.00.R.001.Rev0.Proposed Apartments Development, 20 Helm Street, Mount Pleasant, WA.docx		

Document status and review

Status	Prepared by	Reviewed by	Date issued
Revision 0	Sergio Neves	Fred Verheyde	15 March 2021

Distribution of copies

Status	Electronic	Paper	Issued to
Revision 0	1	0	Anthony Chillino, Pyramid Constructions (WA) Pty Ltd

The undersigned, on behalf of Douglas Partners Pty Ltd, confirm that this document and all attached drawings, logs and test results have been checked and reviewed for errors, omissions and inaccuracies.

	Signature	Date
Author		15 March 2021
Reviewer		15 March 2021

bsi. ISO 9001 Quality Management
FS 604853

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1. Introduction

This report presents the results of a geotechnical investigation undertaken for a proposed apartments development at 20 Helm Street in Mount Pleasant, WA. The investigation was commissioned in a an email dated 25 February 2021 by Anthony Chillino of Pyramid Constructions (WA) Pty Ltd and was undertaken in accordance with Douglas Partners' proposal P201795 dated 19 February 2021.

Based on proposed development plans and cross sections provided by Engenuity Engineering, it is understood that the proposed development will comprise the construction of a four storey building, which will include a single basement with its base at around RL 0 m together with a lift pit (presumed to have a base at about RL – 1.5 m).

The aim of the geotechnical investigation was to assess the sub surface conditions underlying the site and subsequently:

- Identify areas of foundation risk, including areas of uncontrolled fill, compressible layers, potential for liquefaction or any other problematic ground conditions. If encountered, make suggestions in terms of recommended strategies to address any identified risks;
- Recommendations in terms of site preparation, including the possible re-use of existing soils as controlled fill, specification for any imported fill and the removal/treatment of any unsuitable materials encountered;
- Site classification in accordance with AS 2870-2011 and measures that could be adopted to improve this classification, if required. It is emphasised however, that this standard typically applies to buildings up to 2 storeys;
- Earthquake site factor in accordance with AS 1170.4;
- Comment on the excavation conditions, in particular for construction of the proposed basement;
- Recommendations on suitable foundation systems (including individual pad footings, single rafts, piles and piled rafts) and provision of geotechnical parameters for foundation design including allowable bearing pressures for pad and strip footings founded at 0.5 m and 1 m below finished floor levels;
- Estimated short and long-term settlements associated with the recommended founding systems, including potential differential settlements across the proposed structure;
- Recommendations on geotechnical parameters (to a sufficient depth) to enable a detailed pile design in accordance with AS 2159 by a specialist sub-contractor
- Provide advice on suitable ground retention systems and lateral earth pressures for retaining wall design;

- The permeability of the soils and suitability for on-site stormwater disposal using soakwells at a depth of about 3 m below existing site levels; and
- The groundwater level beneath the site at the time of the investigation, and evaluate whether dewatering is likely to be required during construction of the proposed basement and lift pit.

The investigation included the performance of three cone penetration tests (CPT's), the drilling of three boreholes and laboratory testing of selected samples. The details of the field work are presented in this report, together with comments and recommendations on the issues listed above.

2. Site Description

The site comprises no. 20 Helm Street and no. 25 The Esplanade. The site is bounded by Helm Street to the north, The Esplanade to the east and by residential properties elsewhere.

At the time of investigation, the site was occupied by single storey buildings at its centre and associated hardstand and soft landscape areas elsewhere.

Based on publicly available LiDAR data, the surface levels at the site generally grade from the south eastern corner (RL 2.5 m) to the north western corner (RL 3 m). It is emphasised that surface level information available from LiDAR is approximate and should be used with caution.

The Fremantle 1:50,000 Geology Map Sheet indicates that the shallow sub surface conditions beneath the site consist of alluvium comprising sand. Sand derived from the Tamala Limestone is also shown along the western boundary and south western corner of the site.

The Perth Groundwater Atlas indicates that the base of the abovementioned soils is likely to be between RL -15 m and RL -20 m. Furthermore, available data from historical boreholes from the Department of Water and Environmental Regulation (DWER) website (Bores ID reference 61602072 and 61602079 drilled approximately 1.4 km to the southwest of the site), indicates limestone rock (approximately 2 m thick) underlain by sandstone rock from levels of between RL -15 m and RL -17 m.

The Perth Groundwater Atlas (2004) indicates that in May 2003, the groundwater was at around RL 1 m (ie 1 m above the proposed basement level). Given the proximity of the site to the Canning River (approximately 20 m to the east), groundwater levels are anticipated to be near the river water level.

Published acid sulfate soil risk mapping indicates that the proposed development is located within an area of "moderate to low risk of acid sulfate soils within 3 m of natural soil surface". Areas of "no known risk and high to moderate risk of acid sulfate soils within 3 m of natural soil surface", are also shown immediately to the west and east of the site.

3. Field Work Methods

Field work was carried out on 14 March 2019 and comprised:

- Performance of three cone penetration tests (test locations 1 to 3);
- Drilling of three boreholes (04 to 07);
- Perth sand penetrometer (PSP) tests adjacent to the boreholes; and
- Two in situ infiltration tests.

The cone penetration tests were carried out by using a 36 mm diameter instrumented cone with a following 130 mm long friction sleeve attached to rods of the same diameter, pushed continuously at a rate of 2 cm/sec into the soil by hydraulic thrust from a 22 tonne truck rig. Strain gauges in the cone and sleeve measure resistance to penetration and this data allows the assessment of the type and condition of the materials penetrated. The CPTs were pushed to target depths of 10 m at two locations and 20 m at one location. Upon withdrawing the CPT probe, each location was “dipped” to measure the groundwater level.

The boreholes were drilled using a 110 mm hand auger to a depth of 2.6 m, where collapse of the borehole walls below groundwater was experienced. The boreholes were logged in general accordance with AS 1726:2017 by a suitably experienced geotechnical engineer from Douglas Partners. Soil samples were recovered from selected locations for subsequent laboratory testing.

The PSP tests were carried out adjacent to each borehole location in accordance with AS 1289.6.3.3, to assess the in situ density of the shallow soils.

The infiltration tests were undertaken at test locations 4 and 6, and were performed using the falling head method at depths of 2.3 m and 2.4 m below existing ground levels, respectively. The location, depths of testing and results are discussed in detail in Section 4.4.

Test locations were determined using a hand held handheld GPS and are marked on Drawing 1 in Appendix B. Surface elevations at the test locations were obtained from LiDAR data and should be considered approximate. Levels on the CPT traces and borehole logs in Appendix C are quoted relative to AHD.

4. Field Work Results

4.1 Ground Conditions

Detailed CPT traces and borehole logs are presented in Appendix C. Notes defining test methods, descriptive terms and classification methods used are included in Appendix A. The subsurface conditions inferred from the CPTs and encountered in the boreholes are considered to be generally consistent with the geological setting and our expectations.

A summary of the ground conditions encountered at the borehole locations and inferred from the CPTs is given below:

- **Unit 1: SANDY FILL (SAND SP-SM)** – fine to medium grained, grey-brown, with silt, trace rootlets, at all test locations to depths of between 0.5 m and 1 m below existing ground levels. The fill was loose to depths of between 0.5 m and 0.75 m at the test locations.
- **Unit 2A: Alluvial Sandy Soils (SAND SP) –** fine to medium grained, grey-yellow-brown, with various amounts of fines, underlying the fill at all test locations, to a depth of 4.9 m below existing ground levels.

  The sandy alluvium was loose over its entire depth, with the exception of test locations 5 and 6 and the surficial sand at test location 1, which were medium dense (refer to Table 1 below for thicknesses and levels of loose zones).
- **Unit 2B: Alluvial Clayey Soils (Clayey SAND and CLAY) –** inferred underlying the sandy alluvium at test locations 1 to 3, to depths of between 6.3 m and 10.2 m below existing ground levels.

  The clayey alluvium was loose or stiff at test location 1, and medium dense and very stiff to hard at test locations 2 and 3 (refer to Table 1 below for thicknesses and levels of loose clayey alluvium zones).
- **Unit 3: inferred sand derived from Tamala Limestone –** inferred medium dense to very dense, underlying the clayey alluvium at test locations 1 and 2, to a depth of 11.6 m below existing ground level at test location 2.
- **Unit 4: inferred Tamala Limestone –** inferred moderately to well cemented, to a depth of 15 m at test location 2.
- **Unit 5: inferred Limestone or Sand:** weakly cemented limestone or medium dense sand with possibly some clayey sand layers, to a maximum test termination depth of 20 m.

Table 1: Summary of Loose Zones and Approximate Relative Levels

Test Location	Surface Level[1] (m AHD)	Depth of Loose Zone (m)	Level of Loose Zone (m AHD)
1	3.20	6.30	-3.10
2	2.60	4.90	-2.30
3	2.90	4.90	-2.00
4	2.90	>2.50	<0.4
5	2.70	0.45	2.25
6	2.80	0.75	2.05

Note [1]: Surface elevation obtained from available LiDAR data.
- : Not encountered.

4.2 Groundwater

The CPT probe holes were dipped to measure groundwater levels following completion of the testing on 26 February 2021, with the groundwater measurements summarised in Table 2 next page. Groundwater levels observed in the hand auger boreholes drilled on 26 February 2021, are also summarised in Table 2 below. The CPTs and boreholes were immediately backfilled following sampling, which precluded any longer-term monitoring of groundwater levels.

Table 2: Summary of Groundwater Measurements Within the CPTs and Boreholes

Test Location	Surface Level (m AHD)[1]	Groundwater Depth (m)	Approximate Groundwater Level (m AHD)
2	2.6	1.9	0.7
3	2.9	2.2	0.7
4	2.9	2.4	0.5
5	2.7	2.4	0.3
6	2.8	2.4	0.4

Notes: [1] Surface elevation obtained from available LiDAR data.

It should be noted that groundwater levels are potentially affected by various factors such as climatic conditions, land usage and the Canning River water level and will therefore vary with time.

Furthermore, groundwater levels are also affected by long term sea level rise. In accordance with the State Coastal Planning Policy Number 2.6, an allowance for sea level rise of 0.9 m over a 100 year planning timeframe to 2110 should be made.

4.3 Results of Infiltration Testing

Two in-situ infiltration tests were carried out within test locations 4 and 6 using the falling head method, at depths of 2.3 m and 2.4 m below existing ground levels, respectively. Field permeability values were estimated using a method based on Hvorslev (1951) and Ritzema (1994). Permeability can also be estimated from particle size distribution test results from samples taken from the same depths at infiltration test locations, using the Hazen's formula. The Hazen's formula provides an indication of the permeability for clean sand with rounded particle shape in loose conditions, and therefore its applicability to the site conditions should be considered with caution. Table 3 below, summarises the permeability results.

Table 3: Summary of Permeability Analysis

Test Location	Depth (m)	Measured Permeability[1]		Derived Permeability[2]		In situ Conditions of Tested Material
		(m/s)	(m/day)	(m/s)	(m/day)	
4	2.3	5×10^{-6} [3]	0.4[3]	3.2×10^{-4}	>25	Unit 2A: SAND SP, trace silt, loose
6	2.4	2.3×10^{-6} [3]	0.2[3]	2.3×10^{-4}	19	Unit 2A: SAND SP, trace silt, medium dense

Notes: [1]: In situ testing.
[2]: Hazen's formula (assumes sand in loose condition, with rounded sand particles).
[3]: The relatively low field permeability values measured at test locations 4 and 6 are consistent with some weak cementation of the soil observed at a similar depth in adjacent test location 5.

5. Laboratory Testing

A geotechnical laboratory testing programme was carried out by a NATA registered laboratory and comprised the determination of the particle size distribution of two samples and the pH, chloride and sulphate of two samples.

The detailed test report sheets are given in Appendix D, with the results summarised in Tables 4 and 5 below.

Table 4: Summary of Particle Size Distribution Testing

Test Location	Depth (m)	Fines (%)	Sand (%)	Gravel (%)	D_{10}	D_{60}	Material
4	2.5	3	97	0	0.18	0.47	Unit 2A: Sand SP, trace silt
6	2.4	5	95	0	0.15	0.42	

Notes: Fines are particles smaller than 75 µm.
Sand is particles larger than 75 µm and smaller than 2.36 mm.
Gravel is particles larger than 2.36 mm and smaller than 63 mm.
A D_{10} of 0.17 mm means that 10% of the sample particles are less than 0.17 mm.
A D_{60} of 0.26 mm means that 60% of the sample particles are less than 0.26 mm.

Table 5: Results of Laboratory Testing for Soil Aggressivity

Test Location	Depth (m)	Soil Description	Soil Condition[1]	Exposure Classification			
				Concrete		Steel	
				pH	SO4 (mg/kg)	pH	Cl (mg/kg)
4	0.3	Unit 1: FILL/SAND SP-SM with silt	B	7.6	38	7.6	64
6	2.4	Unit 2A: Sand SP, trace silt	A	7.7	<10	7.7	16

Notes: [1]: Soil Type based on guideline presented in AS 2159-2009 and summarise below:
Soil Type A – High permeability soils (e.g. sands and gravels) which are in groundwater.
Soil Type B – Low permeability soils (e.g. silts and clays) or all soils above groundwater.
Scale of aggressivity based on threshold values given in AS 2159-2019

Non-aggressive	Mild	Moderate	Severe	Very Severe

6. Proposed Development

Based on proposed development plans and cross sections provided by Engenuity Engineering, it is understood that the proposed development will comprise the construction of a four storey building, which will include a single basement with its base at around RL 0 m together with a lift pit (presumed to have a base at about RL –1.5 m).

7. Comments

7.1 Site Classification

Due to the occurrence of loose materials, the anticipated classification of the site in its current condition is 'Class P' in accordance with AS 2870-2011.

It should be noted that loose sand extends below the proposed basement excavation level (understood to be RL 0 m, approximately 2.6 m to 3.2 m below the existing site levels) and therefore 'Class P' conditions is also anticipated at proposed basement level.

An amendment of the site classification to 'Class A' in accordance with AS 2870-2011 would require increasing the strength of the encountered loose sand that extends below groundwater to depths up to about 6 m below existing site level, using a ground improvement technique based on either compaction or grout injection, which at this site is likely impractical, financially non-viable or posing unacceptable risks of damage to neighbouring properties. Therefore, proposed structures would require to be fully supported on a piled foundation system.

7.2 Site Seismic Classification and Liquefaction Potential

Ground conditions encountered at the site generally comprise loose sandy soils, becoming medium dense to very dense at depth. The above soils are inferred to overly competent clay, limestone rock and sand. It is suggested that an earthquake design soil sub-class of Ce is appropriate for this site in accordance with AS 1170.4-2007. The Hazard Factor (Z) for the site is 0.09, according to AS1170.4-2007.

Soil liquefaction is a phenomenon whereby a saturated or partially saturated soil substantially loses strength and stiffness in response to an applied stress, usually earthquake shaking or other sudden change in stress condition, causing it to behave like a liquid. This phenomenon is most often observed in saturated, loose sandy soils, such as encountered beneath the site.

The liquefaction risk has been assessed using the results of cone penetration tests, with the assessment indicating a liquefaction risk at all CPT test locations. Liquefaction settlements were assessed using the methodology of Zhang et al (2002) under the assumption of a M_7 earthquake and are assessed to be between 100 mm and 175 mm below proposed basement level across the site.

These settlements could be mitigated if the proposed structure is piled into a competent bearing layer underlying the loose sand.

7.3 Site Preparation

Prior to basement construction, all deleterious material, including demolition rubble resulting from removal of the existing buildings and hardstands, and vegetation, should be stripped and removed from the proposed building envelope.

7.4 Basement Design and Construction

Based on proposed development plans and cross sections provided by Engenuity Engineering, it is understood that the proposed basement will occupy the majority of the site area. As such, due to the anticipated need to excavate up to or close the existing site boundaries, temporary ground retention along these boundaries are likely to form part of the permanent retention system, and thereby the basement walls.

Excavations associated with the basement construction are likely to be undertaken through loose to medium dense granular fill and natural sand. Excavations in these materials should be readily achieved using standard earthmoving equipment (i.e. 8 tonne excavator and larger).

Groundwater levels were recorded as shallow as RL 0.7 m, during the investigation works undertaken when groundwater is likely near its low seasonal level. Groundwater levels at this site are anticipated to be affected by climatic conditions, the Canning River level (located approximately 20 m to the east of the proposed development area) and tides, and therefore higher levels than encountered are anticipated.

Given the above, and based on the proposed basement excavation level, groundwater will impact excavations, and thus it is recommended that provisions for dewatering to a depth of approximately 1 m below the base of any excavations be made, in order to facilitate compaction works.

The most appropriate dewatering methods are considered to be well points, together with the construction of a cut-off wall (see further details in Section 7.4.2) around the perimeter of the proposed basement area to reduce drawdown of groundwater levels in the vicinity of the proposed development area, and thus minimise the risk of excessive settlements and subsequent damage to the existing houses and associated sensitive structures in the vicinity of the site.

Owing to the groundwater level in relation to the proposed basement level, a sealed or tanked basement designed to resist hydrostatic pressure forces will be required.

7.4.1 Batter Slopes

Following demolition of the existing buildings and given the anticipated spacing availability between the proposed basement area and the site boundaries, the formation of batter slopes along the site boundaries to construct the proposed basement is unlikely to be feasible.

For other excavations, it is recommended that batter slopes above groundwater not steeper than 1.5H:1V (horizontal : vertical) be adopted, for temporary excavations not deeper than 3 m in sand materials. The above recommended batter angle should be re-assessed if loads are to be applied near the top of the batter.

It is also emphasised that the abovementioned batter angle is not applicable if water emanates from the excavation slopes (see comments in Section 7.4 above with regards to groundwater and dewatering requirements). In such conditions, there is potential for instability no matter how flat the batter angle.

If loads are applied at the top of the batter (for example, excavated soil or equipment), or if there is any groundwater influence, then a site specific assessment of stability should be undertaken, or consideration could be given to adopt ground support systems as described in Section 7.4.2 below.

7.4.2 Ground Support Systems

Excavation for the currently proposed basement is anticipated to require construction of a perimeter ground retention system along site boundaries.

A secant piled wall is recommended for construction of a perimeter ground retention system. Continuous Flight Auger piles (CFA) offer many advantages over conventional bored piles to construct the walls given the former do not require the use of temporary casing or drilling fluid as part of the piling operations.

Consideration was given to the use of sheet piles. However, sheet piles installation generates vibration, and thus the risk of excessive settlement under existing houses and associated sensitive structures foundations is considered to be high during sheet piles installation, and thus their use is not recommended. Consideration was also given to the use of contiguous piles, however because these piles are installed at close spacings typically with nominal gaps between the piles or just touching, there is no continuous barrier to prevent ingress of infiltrating water or migration of soil into the basement. Thus, the use of contiguous piles is not recommended.

7.4.3 Lateral Earth Pressures

The lateral earth pressures acting on the basement walls, depend on several factors including the wall type, sequence of construction and the permissible ground movements behind the walls.

Design parameters for the design of temporary and permanent retaining structures are suggested in Table 6 next page.

Table 6: Suggested Soil Parameters for Retaining Wall Design

Soil Type	Design Depth Range below existing ground level (m)	Drained Angle of Friction Φ' (degrees)	Undrained Shear Strength C_u (kPa)	Soil Unit Weight above Water γ (kN/m³)	Soil Unit Weight below Water γ (kN/m³)	Coefficient of Active Earth Pressure		
						K_a	K_0	K_p [1]
Unit 1 and 2A - Fill and Natural Sand (Loose)	0 to 5	30	0	18	8	0.33	0.5	3
Unit 2B – Clay and Clayey Sand (stiff/loose and stronger)	5 to 10	20	75	20	10	0.3	0.5	2
Unit 3, 4 and 5 – Sand and Limestone (Medium dense and denser)	> 10	34	0	20	10	0.28	0.44	3.5

Note [1]: Ultimate values that need to incorporate a factor of safety of at least two to derive a design value.

A large diameter rigid pile wall or a strutted pile wall is recommended to minimise displacement behind the wall and damage to adjacent neighbouring properties. For strutted pile walls restrained at the top, the distribution of earth pressures tends to be uniform with depth. Design earth pressures are presented below for two situations depending on whether or not movement sensitive elements are within the influence zone behind the walls.

<u>Adjacent buildings and movement sensitive assets</u>

For piles, a uniform pressure distribution over the full height of the wall can be adopted and calculated as:

a) $\sigma_H = 5H + 0.5\,q$ (kPa)

σ_H = lateral earth pressure
H = height of wall in metres
q = surcharge pressure

<u>No adjacent buildings or movement sensitive assets</u>

For piles, the earth pressure can be calculated as:

b) $\sigma_H = 4H + 0.4\,q$ (kPa)

The walls should be designed for hydrostatic pressures, acting below the long term design water level, with an allowance for transient excess pressures following periods of heavy rainfall.

In addition, footing loads from adjacent structures can be calculated using a coefficient of earth pressure, K, of 0.5 and 0.4 for cases a) and b), respectively.

The passive resistance for the embedded wall section can be assessed using an earth pressure coefficient, K_p, of 3 for loose sands. This is an ultimate value and would need to incorporate a factor of safety to limit lateral movements.

7.4.4 Monitoring of Adjacent Buildings

Construction of the basement could lead to settlement outside the excavation and inwards movement of the walls caused by a combination of excavation rebound and soil relaxation. Therefore, dilapidation survey of adjacent buildings is recommended prior to any construction works.

7.5 Re-use of In Situ Material

The fill and naturally occurring sand excavated from the site should be suitable for re-use as structural fill, provided it is free from organic material and particles greater than 150 mm in size.

It is recommended that re-used sandy soils be placed in loose lift thickness within 2% of its optimum moisture content with each layer compacted to achieve a dry density ratio of not less than 95% relative to modified compaction.

Compaction control of sand could be carried out using a Perth Sand Penetrometer in accordance with test method AS 1289.6.3.3, after the relationship between penetrometer blow counts and soil density is established.

It is recommended that verification of the compaction works be undertaken by an experienced geotechnical engineer.

7.6 Foundation Systems

7.6.1 Foundation System Selection

A piled foundation system is considered suitable for the proposed building. In particular, owing to the occurrence of loose sand and risk of liquefaction below proposed basement level, a foundation system that would include high level footings or a rigid raft is not considered suitable at this site.

Continuous flight auger piles (CFA), bored piles or barrettes (rectangular piles) are considered suitable for the site conditions. Unless continuous segmental casing is used, bored piles would need to be drilled under a drilling fluid, with the associated need for batching plant and de-sanding equipment where bentonite mud is used. The same applies to barrettes.

CFA piles would not need such equipment and are installed essentially without vibration. It is not possible, however, to obtain high quality logs of the excavated shafts or to identify the material at pile toe level with CFA piles until the auger is withdrawn and the pile shafts formed. Accordingly, these piles are typically installed to pre-determined levels. Instrumentation of drilling parameters such as torque and rate of penetration and volume of injected concrete will provide a degree of assurance of the materials in which the pile is formed. The depth to which reinforcement cages can be installed in CFA piles is typically limited to about 15 m to 20 m depending on the concrete mix design, which will need to be taken into account if consideration is given to use CFA piles as tension piles. Further advice in this regard should be sought from specialist piling contractors, based on their available piling equipment.

The above pile types involve soil excavation to target founding level and hence would produce spoil that needs to be handled and disposed of in a suitable manner.

7.6.2 CFA, Bored and Barrette Piles

For a project of this nature, the standard practice is for specialist piling contractors to submit their own designs. Accordingly, this geotechnical report should be made available to selected piling contractors in order for them to nominate pile capacities, target founding levels and estimated settlements. The pile design should be carried out in accordance with the minimum requirements of Australian Standard AS 2159 – 2009 Piling, Design and Installation.

It is recommended that the piles should be founded at least in the dense to very dense materials of Unit 3, resulting in anticipated pile lengths of at least 7 m below basement level (10 m below existing ground level). The piles can be designed to derive their compression capacity from a combination of shaft resistance and end bearing resistance.

The preliminary parameters given in Table 7 below are suggested for preliminary assessment of the geotechnical strength for piles in axial compression. CFA piles can generally achieve higher shaft resistance values compared to bored piles although this depends on the contractor's experience to a large degree. For present purposes, the quoted shaft resistance unit stresses have not been differentiated on the basis of pile type.

Note the skin friction stresses quoted are average values and may need to be adjusted for pile length and specific designs. To develop the end bearing resistances, piles should be embedded at least 4 pile diameters into the respective strata.

Table 7: Preliminary Pile Design Parameters for CFA, Bored and Barrette Piles

Inferred Unit (see Section 4.1)	Design Depth Range of Unit (m below existing ground level)	Average Ultimate Skin Friction (kPa)	Ultimate End Bearing (kPa)
Units 1 and 2A - Sand (loose)	0 to 5	0[1]	NA
Unit 2B – Alluvial Clayey Soils (loose / stiff and stronger)	5 to 10	40	NA
Units 3, 4 and 5 – Sand and Limestone (various strengths and cementation)	>10	90[2]	2,500[2]

Note: [1] Design shaft friction into loose sand is recommended to be zero, owing to the liquefaction risk of this layer.
[2] These values consider the reduced thickness of the limestone and variable strength of the geological units.

Pile shaft friction in soils should be assumed to be zero within 1.5 times pile diameter of the finished ground surface and within the encountered loose sand below groundwater owing to possible ground disturbance effects during construction and liquefaction risk under earthquake of the loose sand.

The skin friction for tension (or uplift) loading could be taken as 70% of the shaft values given for compression.

A relatively high redundancy piling system is recommended for piles ending into Units 3, 4 or 5, owing to the variable strength of these units, that might also include lower strength materials than encountered during the investigation.

The pile capacities derived using the design parameters suggested in the previous page are ultimate values, that require to be multiplied by a strength reduction factor discussed in the following section to derive design values.

7.6.3 Geotechnical Strength Reduction Factor

Australian Standard AS 2159-2009 provides the minimum requirements for the design of piled footings based on a limit state approach. Accordingly, the calculated ultimate pile capacity depends on the selected geotechnical strength reduction factor. Selection of the geotechnical strength reduction factor (Φg) in accordance with AS 2159 Table 4.3.2 (A) is based upon a series of individual risk ratings and the final value of Φg depends on the following factors:

a) Site: the type, quantity and quality of testing.

b) Design: design methods and parameter selection.

c) Installation: construction control and monitoring.

d) Pile testing regime testing benefit factor based on percentage of piles tested and the type of testing. If some testing is carried out, an increase in the value of Φg may be possible depending on the type and extent of the testing. It is noted that Table 8.2.4(B) of AS 2159-2009 requires that 5% to 15% of piles should be subject to integrity testing if the value of Φg adopted by the structural designer exceeds 0.4.

e) Redundancy: whether other piles can take up load if a given pile settles or fails.

Of the above factors, Douglas Partners can only comment directly upon the site factors under a). The pile designer must determine the individual risk factors b) - e) with knowledge of the pile construction specification that will be applied to the works.

The assessed AS 2159 risk factors assigned by Douglas Partners to the site conditions for the investigation is ‘2’, ‘4’ and ‘3’ respectively for each of the three individual risk factors in item a) above.

As the adopted reduction factor is a function of the above combined effect, it is therefore best determined at detailed design. For preliminary design, a Φ_g value of 0.52 is suggested assuming that pile testing will be undertaken at the site.

7.6.4 Settlement of Piles

The settlement of an individual pile founded in dense or denser sand is expected to be in the order of 0.5% to 1% of pile diameter at normal working loads.

Larger settlement should be anticipated under pile groups but would require a separate analysis once details on pile loads and pile arrangement are known.

7.6.5 Pile Testing

The Piling Standard (AS 2159) encourages pile load testing by allowing the adoption of higher pile capacities if piles are tested.

The Standard requires that the as-constructed piles should be subjected to sonic integrity testing (SIT) to confirm the length, continuity and overall integrity of the piles for the site conditions and the piling plant employed. It is considered that a minimum rate of integrity testing of 50% should be adopted for piling rigs with full QA monitoring instrumentation, as defined in the Piling Code. It is recommended that CFA piles without full instrumentation should not be considered for the works.

The most cost effective means of verifying the pile design capacities is to use dynamic testing techniques coupled with CAPWAP or equivalent wave matching analysis. Guidelines on the frequency of pile testing are presented in AS 2159. In order to utilise upper bound capacity resistance factors, it is suggested that the frequency of pile testing should be towards the upper end of the recommended range. The testing of bored and CFA piles would require the use of heavy drop masses in order to demonstrate that the minimum resistances can be mobilised. Additionally, it would be necessary to cast an integral above ground reinforced section to allow mounting of the instrumentation. Further, the amount of steel in the test piles may need to be adjusted to ensure that the induced tensile stresses do not exceed permissible limits.

The dynamic pile testing described above generates vibrations. As a result, existing houses and possible sensitive structures in the vicinity of the site should be considered prior to undertaking any pile testing, and may preclude any pile dynamic testing at the site. If this is the case, it is suggested that piles static load testing be undertaken in place of piles dynamic testing.

7.7 Working Platform

Owing to the occurrence of loose sand, a suitable working platform is anticipated to be required to support piling cranes and ancillary construction plant. It is recommended that allowance be made to install a granular platform as this will improve site trafficability. Comments on the requirement, design, construction and maintenance of working platforms can be addressed once the type of equipment and loads are known.

Piling activities will need to be coordinated with the bulk excavation works with regard to the level the piling rig will be operating from.

7.8 Soil Aggressivity

Results of the testing performed on a soil sample for electric conductivity, pH, chloride and sulphate ion concentration were compared with the exposure classifications in AS 2159 – 2009. The results indicate the following classifications:

- For concrete: a non-aggressive classification in test location 4 and for soils above groundwater and a mild classification for soils below groundwater in test location 6; and

- For steel: a non-aggressive classification for soils above and below groundwater in test locations 4 and 6.

7.9 Soil Permeability and Stormwater Disposal

As discussed in Section 4.1, the shallow ground conditions beneath the site generally comprise loose to medium dense sand fill and natural sand.

Results of the permeability testing summarised in Section 4.3 indicates field permeability values of the order of 0.2 m/day to 0.4 m/day for the sand near proposed basement levels. These relatively low values for sand are consistent with some weak cementation of the soil at this level and are considered representative at proposed basement level (ie below groundwater).

It should be noted that groundwater level is above proposed basement level and will therefore preclude the use of infiltration systems from the base of the proposed basement (that will likely be tanked).

However, the use of soakwells or other infiltration systems should be suitable at shallower depths, where sand of greater presumptive permeability is expected.

7.10 Suggested Geotechnical Testing for Supplementary Geotechnical Investigation

Based on the information collected and access limitations as part of the ground investigation works, the ground conditions suggest that the following items should be addressed during a supplementary ground investigation in order to suitably assess the ground conditions within the proposed development area:

- Further assessment of the loose ground conditions, and assessment of the lateral extent of these, within areas not investigated as part of the investigation works; and
- Further information on the depth to bedrock and an assessment of its conditions may be required, if preliminary foundation design suggest that a piled foundation system is required to be founded into the bedrock material.
- Further assessment of shallow soil permeability for possible shallow stormwater infiltration systems.
- Assess soil and groundwater aggressivity at representative depths for piles.

Table 8 next page includes the minimum and possible supplementary testing to address the above items.

Table 8: Suggested Supplementary Geotechnical Testing

Items To Address	Proposed Testing
To assess the loose conditions within the areas not investigated as part of the investigation works.	2 CPTs – 10 m and 20 m depth or prior refusal.
To assess the depth to bedrock and its conditions within the proposed building footprint, if required (see comment in the text above).	1 borehole to 40 m depth, with installation of one standpipe.
To assess shallow soil permeability.	3 infiltration tests at the locations and depths of proposed infiltration systems.
To assess soil and groundwater aggressivity at depth.	Laboratory testing on samples collected from the borehole sand standpipe.

8. References

1. AS 2870 (2011). *Residential Slabs and Footings.* Standards Australia.
2. AS 1289 (2000). *Methods of Testing Soils for Engineering Purposes.* Standards Australia.
3. AS 1289.6.3.3 (1999). *Soil Strength and Consolidation Tests-Determination of the Penetration Resistance of a Soil – Perth Sand Penetrometer Test.* Standards Australia.
4. AS 1726 (2017). *Geotechnical Site Investigation.* Standards Australia.
5. AS 3798 (2007). *Guidelines on Earthworks for Commercial and Residential Developments.* Standards Australia.
6. AS 2159 (2009). *Piling – Design and Installation.* Standards Australia.
7. Department of Environment, *Perth Groundwater Atlas, Second Edition, December 2004*.
8. G. Zhang, PK Robertson and RWI Brachman (2002). *Estimating liquefaction-induced ground settlements from CPT for level ground*.

9. Limitations

Douglas Partners has prepared this report for the proposed apartments development at 20 Helm Street, Mount Pleasant, WA in accordance with Douglas Partners' proposal dated 19 February 2021 and acceptance received from Anthony Chillino of Pyramid Constructions (WA) Pty Ltd in an email dated 25 February 2021. The work was carried out under Douglas Partners' Conditions of Engagement. This report is provided for the exclusive use of Pyramid Constructions (WA) Pty Ltd for this project only and for the purposes as described in the report. It should not be used by or relied upon for other projects or purposes on the same or other site or by a third party. Any party so relying upon this report beyond its exclusive use and purpose as stated above, and without the express written consent of Douglas Partners, does so entirely at its own risk and without recourse to Douglas Partners for any loss or damage. In preparing this report Douglas Partners has necessarily relied upon information provided by the client and/or their agents.

The results provided in the report are indicative of the sub-surface conditions on the site only at the specific sampling and/or testing locations, and then only to the depths investigated and at the time the work was carried out. Sub-surface conditions can change abruptly due to variable geological processes and also as a result of human influences. Such changes may occur after Douglas Partners' field testing has been completed.

Douglas Partners' advice is based upon the conditions encountered during this investigation. The accuracy of the advice provided by Douglas Partners in this report may be affected by undetected variations in ground conditions across the site between and beyond the sampling and/or testing locations. The advice may also be limited by budget constraints imposed by others or by site accessibility.

The assessment of atypical safety hazards arising from this advice is restricted to the geotechnical components set out in this report and based on known project conditions and stated design advice and assumptions. While some recommendations for safe controls may be provided, detailed 'safety in design' assessment is outside the current scope of this report and requires additional project data and assessment.

This report must be read in conjunction with all of the attached and should be kept in its entirety without separation of individual pages or sections. Douglas Partners cannot be held responsible for interpretations or conclusions made by others unless they are supported by an expressed statement, interpretation, outcome or conclusion stated in this report.

This report, or sections from this report, should not be used as part of a specification for a project, without review and agreement by Douglas Partners. This is because this report has been written as advice and opinion rather than instructions for construction.

Douglas Partners Pty Ltd

Appendix A

About This Report

Introduction

These notes have been provided to amplify DP's report in regard to classification methods, field procedures and the comments section. Not all are necessarily relevant to all reports.

DP's reports are based on information gained from limited subsurface excavations and sampling, supplemented by knowledge of local geology and experience. For this reason, they must be regarded as interpretive rather than factual documents, limited to some extent by the scope of information on which they rely.

Copyright

This report is the property of Douglas Partners Pty Ltd. The report may only be used for the purpose for which it was commissioned and in accordance with the Conditions of Engagement for the commission supplied at the time of proposal. Unauthorised use of this report in any form whatsoever is prohibited.

Borehole and Test Pit Logs

The borehole and test pit logs presented in this report are an engineering and/or geological interpretation of the subsurface conditions, and their reliability will depend to some extent on frequency of sampling and the method of drilling or excavation. Ideally, continuous undisturbed sampling or 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 boreholes and test pits 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 boreholes or pits, the frequency of sampling, and the possibility of other than 'straight line' variations between the test locations.

Groundwater

Where groundwater levels are measured in boreholes there are several potential problems, namely:

- In low permeability soils groundwater may enter the hole very slowly or perhaps not at all during the time the hole 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 weather changes. They may not be the same at the time of construction as are indicated in the report; and
- The use of water or mud as a drilling fluid will mask any groundwater inflow. Water has to be blown out of the hole and drilling mud must first be washed out of the hole if water measurements 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.

Reports

The report has been prepared by qualified personnel, is based on the information obtained from field and laboratory testing, and has been undertaken to current engineering standards of interpretation and analysis. Where the report has been prepared for a specific design proposal, the information and interpretation may not be relevant if the design proposal is changed. If this happens, DP will be pleased to review the report and the sufficiency of the investigation work.

Every care is taken with the report as it relates to interpretation of subsurface conditions, discussion of geotechnical and environmental aspects, and recommendations or suggestions for design and construction. However, DP cannot always anticipate or assume responsibility for:

- Unexpected variations in ground conditions. The potential for this will depend partly on borehole or pit spacing and sampling frequency;
- Changes in policy or interpretations of policy by statutory authorities; or
- The actions of contractors responding to commercial pressures.

If these occur, DP will be pleased to assist with investigations or advice to resolve the matter.

About this Report

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

Information for Contractual Purposes
Where information obtained from this report is provided for tendering purposes, it is recommended 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. DP would be pleased to assist in this regard and/or to make additional report copies available for contract purposes at a nominal charge.

Site Inspection
The company will always be pleased to provide engineering inspection services for geotechnical and environmental aspects of work to which this report is related. This could range from a site visit to confirm that conditions exposed are as expected, to full time engineering presence on site.

Cone Penetration Tests

Douglas Partners

Introduction

The Cone Penetration Test (CPT) is a sophisticated soil profiling test carried out in-situ. A special cone shaped probe is used which is connected to a digital data acquisition system. The cone and adjoining sleeve section contain a series of strain gauges and other transducers which continuously monitor and record various soil parameters as the cone penetrates the soils.

The soil parameters measured depend on the type of cone being used, however they always include the following basic measurements

- Cone tip resistance q_c
- Sleeve friction f_s
- Inclination (from vertical) i
- Depth below ground z

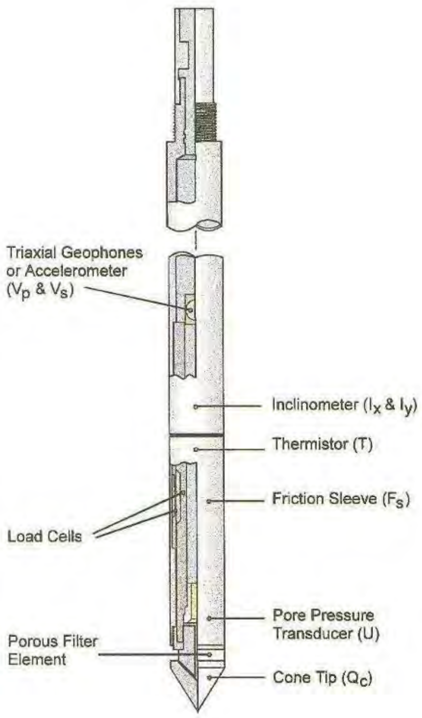


Figure 1: Cone Diagram

The inclinometer in the cone enables the verticality of the test to be confirmed and, if required, the vertical depth can be corrected.

The cone is thrust into the ground at a steady rate of about 20 mm/sec, usually using the hydraulic rams of a purpose built CPT rig, or a drilling rig. The testing is carried out in accordance with the Australian Standard AS1289 Test 6.5.1.

Figure 2: Purpose built CPT rig

The CPT can penetrate most soil types and is particularly suited to alluvial soils, being able to detect fine layering and strength variations. With sufficient thrust the cone can often penetrate a short distance into weathered rock. The cone will usually reach refusal in coarse filling, medium to coarse gravel and on very low strength or better rock. Tests have been successfully completed to more than 60 m.

Types of CPTs

Douglas Partners (and its subsidiary GroundTest) owns and operates the following types of CPT cones:

Type	Measures
Standard	Basic parameters (q_c, f_s, i & z)
Piezocone	Dynamic pore pressure (u) plus basic parameters. Dissipation tests estimate consolidation parameters
Conductivity	Bulk soil electrical conductivity (σ) plus basic parameters
Seismic	Shear wave velocity (V_s), compression wave velocity (V_p), plus basic parameters

Strata Interpretation

The CPT parameters can be used to infer the Soil Behaviour Type (SBT), based on normalised values of cone resistance (Qt) and friction ratio (Fr). These are used in conjunction with soil classification charts, such as the one below (after Robertson 1990)

Cone Penetration Tests

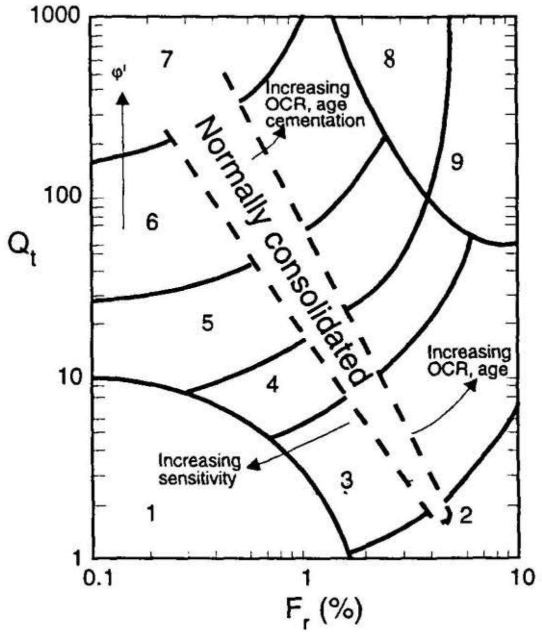


Figure 3: Soil Classification Chart

DP's in-house CPT software provides computer aided interpretation of soil strata, generating soil descriptions and strengths for each layer. The software can also produce plots of estimated soil parameters, including modulus, friction angle, relative density, shear strength and over consolidation ratio.

DP's CPT software helps our engineers quickly evaluate the critical soil layers and then focus on developing practical solutions for the client's project.

Engineering Applications

There are many uses for CPT data. The main applications are briefly introduced below:

Settlement
CPT provides a continuous profile of soil type and strength, providing an excellent basis for settlement analysis. Soil compressibility can be estimated from cone derived moduli, or known consolidation parameters for the critical layers (eg. from laboratory testing). Further, if pore pressure dissipation tests are undertaken using a piezocone, in-situ consolidation coefficients can be estimated to aid analysis.

Pile Capacity
The cone is, in effect, a small scale pile and, therefore, ideal for direct estimation of pile capacity. DP's in-house program ConePile can analyse most pile types and produces pile capacity versus depth plots. The analysis methods are based on proven static theory and empirical studies, taking account of scale effects, pile materials and method of installation. The results are expressed in limit state format, consistent with the Piling Code AS2159.

Dynamic or Earthquake Analysis
CPT and, in particular, Seismic CPT are suitable for dynamic foundation studies and earthquake response analyses, by profiling the low strain shear modulus G_0. Techniques have also been developed relating CPT results to the risk of soil liquefaction.

Other Applications
Other applications of CPT include ground improvement monitoring (testing before and after works), salinity and contaminant plume mapping (conductivity cone), preloading studies and verification of strength gain.

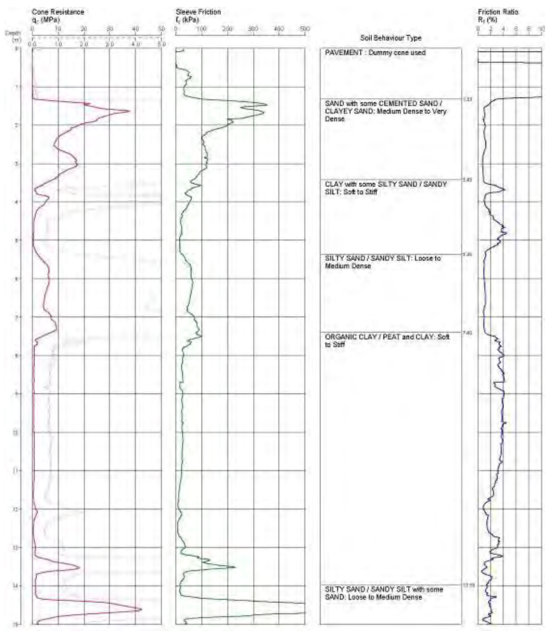


Figure 4: Sample Cone Plot

Soil Descriptions

Douglas Partners

Description and Classification Methods

The methods of description and classification of soils and rocks used in this report are generally based on Australian Standard AS1726:2017, Geotechnical Site Investigations. In general, the descriptions include strength or density, colour, structure, soil or rock type and inclusions.

Soil Types

Soil types are described according to the predominant particle size, qualified by the grading of other particles present:

Type	Particle size (mm)
Boulder	>200
Cobble	63 - 200
Gravel	2.36 - 63
Sand	0.075 - 2.36
Silt	0.002 - 0.075
Clay	<0.002

The sand and gravel sizes can be further subdivided as follows:

Type	Particle size (mm)
Coarse gravel	19 - 63
Medium gravel	6.7 - 19
Fine gravel	2.36 – 6.7
Coarse sand	0.6 - 2.36
Medium sand	0.21 - 0.6
Fine sand	0.075 - 0.21

Definitions of grading terms used are:

- Well graded - a good representation of all particle sizes
- Poorly graded - an excess or deficiency of particular sizes within the specified range
- Uniformly graded - an excess of a particular particle size
- Gap graded - a deficiency of a particular particle size with the range

The proportions of secondary constituents of soils are described as follows:

In fine grained soils (>35% fines)

Term	Proportion of sand or gravel	Example
And	Specify	Clay (60%) and Sand (40%)
Adjective	>30%	Sandy Clay
With	15 – 30%	Clay with sand
Trace	0 - 15%	Clay with trace sand

In coarse grained soils (>65% coarse)
- with clays or silts

Term	Proportion of fines	Example
And	Specify	Sand (70%) and Clay (30%)
Adjective	>12%	Clayey Sand
With	5 - 12%	Sand with clay
Trace	0 - 5%	Sand with trace clay

In coarse grained soils (>65% coarse)
- with coarser fraction

Term	Proportion of coarser fraction	Example
And	Specify	Sand (60%) and Gravel (40%)
Adjective	>30%	Gravelly Sand
With	15 - 30%	Sand with gravel
Trace	0 - 15%	Sand with trace gravel

The presence of cobbles and boulders shall be specifically noted by beginning the description with ‘Mix of Soil and Cobbles/Boulders’ with the word order indicating the dominant first and the proportion of cobbles and boulders described together.

Soil Descriptions

Cohesive Soils

Cohesive soils, such as clays, are classified on the basis of undrained shear strength. The strength may be measured by laboratory testing, or estimated by field tests or engineering examination. The strength terms are defined as follows:

Description	Abbreviation	Undrained shear strength (kPa)
Very soft	VS	<12
Soft	S	12 - 25
Firm	F	25 - 50
Stiff	St	50 - 100
Very stiff	VSt	100 - 200
Hard	H	>200
Friable	Fr	-

Cohesionless Soils

Cohesionless soils, such as clean sands, are classified on the basis of relative density, generally from the results of standard penetration tests (SPT), cone penetration tests (CPT) or dynamic penetrometers (PSP). The relative density terms are given below:

Relative Density	Abbreviation	Density Index (%)
Very loose	VL	<15
Loose	L	15-35
Medium dense	MD	35-65
Dense	D	65-85
Very dense	VD	>85

Soil Origin

It is often difficult to accurately determine the origin of a soil. Soils can generally be classified as:

- Residual soil - derived from in-situ weathering of the underlying rock;
- Extremely weathered material – formed from in-situ weathering of geological formations. Has soil strength but retains the structure or fabric of the parent rock;
- Alluvial soil – deposited by streams and rivers;
- Estuarine soil – deposited in coastal estuaries;
- Marine soil – deposited in a marine environment;
- Lacustrine soil – deposited in freshwater lakes;
- Aeolian soil – carried and deposited by wind;
- Colluvial soil – soil and rock debris transported down slopes by gravity;
- Topsoil – mantle of surface soil, often with high levels of organic material.
- Fill – any material which has been moved by man.

Moisture Condition – Coarse Grained Soils

For coarse grained soils the moisture condition should be described by appearance and feel using the following terms:

- Dry (D) Non-cohesive and free-running.
- Moist (M) Soil feels cool, darkened in colour.

  Soil tends to stick together.

  Sand forms weak ball but breaks easily.
- Wet (W) Soil feels cool, darkened in colour.

  Soil tends to stick together, free water forms when handling.

Moisture Condition – Fine Grained Soils

For fine grained soils the assessment of moisture content is relative to their plastic limit or liquid limit, as follows:

- 'Moist, dry of plastic limit' or 'w <PL' (i.e. hard and friable or powdery).
- 'Moist, near plastic limit' or 'w ≈ PL (i.e. soil can be moulded at moisture content approximately equal to the plastic limit).
- 'Moist, wet of plastic limit' or 'w >PL' (i.e. soils usually weakened and free water forms on the hands when handling).
- 'Wet' or 'w ≈LL' (i.e. near the liquid limit).
- 'Wet' or 'w >LL' (i.e. wet of the liquid limit).

Rock Strength

Rock strength is defined by the Unconfined Compressive Strength and it refers to the strength of the rock substance and not the strength of the overall rock mass, which may be considerably weaker due to defects.

The Point Load Strength Index $Is_{(50)}$ is commonly used to provide an estimate of the rock strength and site specific correlations should be developed to allow UCS values to be determined. The point load strength test procedure is described by Australian Standard AS4133.4.1-2007. The terms used to describe rock strength are as follows:

Strength Term	Abbreviation	Unconfined Compressive Strength MPa	Point Load Index * $Is_{(50)}$ MPa
Very low	VL	0.6 - 2	0.03 - 0.1
Low	L	2 - 6	0.1 - 0.3
Medium	M	6 - 20	0.3 - 1.0
High	H	20 - 60	1 - 3
Very high	VH	60 - 200	3 - 10
Extremely high	EH	>200	>10

* Assumes a ratio of 20:1 for UCS to $Is_{(50)}$. It should be noted that the UCS to $Is_{(50)}$ ratio varies significantly for different rock types and specific ratios should be determined for each site.

Degree of Weathering

The degree of weathering of rock is classified as follows:

Term	Abbreviation	Description
Residual Soil	RS	Material is weathered to such an extent that it has soil properties. Mass structure and material texture and fabric of original rock are no longer visible, but the soil has not been significantly transported.
Extremely weathered	XW	Material is weathered to such an extent that it has soil properties. Mass structure and material texture and fabric of original rock are still visible
Highly weathered	HW	The whole of the rock material is discoloured, usually by iron staining or bleaching to the extent that the colour of the original 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	MW	The whole of the rock material is discoloured , usually by iron staining or bleaching to the extent that the colour of the original rock is not recognisable, but shows little or no change of strength 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	No signs of decomposition or staining.
Note: If HW and MW cannot be differentiated use DW (see below)		
Distinctly weathered	DW	Rock strength usually changed by weathering. The rock may be highly discoloured, usually by iron staining. Porosity may be increased by leaching or may be decreased due to deposition of weathered products in pores.

Rock Descriptions

Degree of Fracturing

The following classification applies to the spacing of natural fractures in diamond drill cores. It includes bedding plane partings, joints and other defects, but excludes drilling breaks.

Term	Description
Fragmented	Fragments of <20 mm
Highly Fractured	Core lengths of 20-40 mm with occasional fragments
Fractured	Core lengths of 30-100 mm with occasional shorter and longer sections
Slightly Fractured	Core lengths of 300 mm or longer with occasional sections of 100-300 mm
Unbroken	Core contains very few fractures

Rock Quality Designation

The quality of the cored rock can be measured using the Rock Quality Designation (RQD) index, defined as:

$$\text{RQD \%} = \frac{\text{cumulative length of 'sound' core sections} \geq 100 \text{ mm long}}{\text{total drilled length of section being assessed}}$$

where 'sound' rock is assessed to be rock of low strength or stronger. The RQD applies only to natural fractures. If the core is broken by drilling or handling (i.e. drilling breaks) then the broken pieces are fitted back together and are not included in the calculation of RQD.

Stratification Spacing

For sedimentary rocks the following terms may be used to describe the spacing of bedding partings:

Term	Separation of Stratification Planes
Thinly laminated	< 6 mm
Laminated	6 mm to 20 mm
Very thinly bedded	20 mm to 60 mm
Thinly bedded	60 mm to 0.2 m
Medium bedded	0.2 m to 0.6 m
Thickly bedded	0.6 m to 2 m
Very thickly bedded	> 2 m

Symbols & Abbreviations

Douglas Partners

Introduction

These notes summarise abbreviations commonly used on borehole logs and test pit reports.

Drilling or Excavation Methods

C	Core drilling
R	Rotary drilling
SFA	Spiral flight augers
NMLC	Diamond core - 52 mm dia
NQ	Diamond core - 47 mm dia
HQ	Diamond core - 63 mm dia
PQ	Diamond core - 81 mm dia

Water

▷	Water seep
▽	Water level

Sampling and Testing

A	Auger sample
B	Bulk sample
D	Disturbed sample
E	Environmental sample
U_{50}	Undisturbed tube sample (50mm)
W	Water sample
pp	Pocket penetrometer (kPa)
PID	Photo ionisation detector
PL	Point load strength Is(50) MPa
S	Standard Penetration Test
V	Shear vane (kPa)

Description of Defects in Rock

The abbreviated descriptions of the defects should be in the following order: Depth, Type, Orientation, Coating, Shape, Roughness and Other. Drilling and handling breaks are not usually included on the logs.

Defect Type

B	Bedding plane
Cs	Clay seam
Cv	Cleavage
Cz	Crushed zone
Ds	Decomposed seam
F	Fault
J	Joint
Lam	Lamination
Pt	Parting
Sz	Sheared Zone
V	Vein

Orientation

The inclination of defects is always measured from the perpendicular to the core axis.

h	horizontal
v	vertical
sh	sub-horizontal
sv	sub-vertical

Coating or Infilling Term

cln	clean
co	coating
he	healed
inf	infilled
stn	stained
ti	tight
vn	veneer

Coating Descriptor

ca	calcite
cbs	carbonaceous
cly	clay
fe	iron oxide
mn	manganese
slt	silty

Shape

cu	curved
ir	irregular
pl	planar
st	stepped
un	undulating

Roughness

po	polished
ro	rough
sl	slickensided
sm	smooth
vr	very rough

Other

fg	fragmented
bnd	band
qtz	quartz

Symbols & Abbreviations

Graphic Symbols for Soil and Rock

General

- Asphalt
- Road base
- Concrete
- Filling

Soils

- Topsoil
- Peat
- Clay
- Silty clay
- Sandy clay
- Gravelly clay
- Shaly clay
- Silt
- Clayey silt
- Sandy silt
- Sand
- Clayey sand
- Silty sand
- Gravel
- Sandy gravel
- Cobbles, boulders
- Talus

Sedimentary Rocks

- Boulder conglomerate
- Conglomerate
- Conglomeratic sandstone
- Sandstone
- Siltstone
- Laminite
- Mudstone, claystone, shale
- Coal
- Limestone

Metamorphic Rocks

- Slate, phyllite, schist
- Gneiss
- Quartzite

Igneous Rocks

- Granite
- Dolerite, basalt, andesite
- Dacite, epidote
- Tuff, breccia
- Porphyry

Sampling

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

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

Undisturbed samples are taken by pushing a thin-walled sample tube into the soil and withdrawing it to obtain a sample of the soil 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.

Test Pits

Test pits are usually excavated with a backhoe or an excavator, allowing close examination of the in-situ soil if it is safe to enter into the pit. The depth of excavation is limited to about 3 m for a backhoe and up to 6 m for a large excavator. A potential disadvantage of this investigation method is the larger area of disturbance to the site.

Large Diameter Augers

Boreholes can be drilled using a rotating plate or short spiral auger, generally 300 mm or larger in diameter commonly mounted on a standard piling rig. The cuttings are returned to the surface at intervals (generally 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 samples.

Continuous Spiral Flight Augers

The borehole 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 sands above the water table. Samples are returned to the surface, or may be collected after withdrawal of the auger flights, but they are disturbed and may be mixed with soils from the sides of the hole. Information from the drilling (as distinct from specific sampling by SPTs or undisturbed samples) is of relatively low reliability, due to the remoulding, possible mixing or softening of samples by groundwater.

Non-core Rotary Drilling

The borehole is advanced using a rotary bit, with water or drilling mud 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 the rate of penetration. Where drilling mud is used this can mask the cuttings and reliable identification is only possible from separate sampling such as SPTs.

Continuous Core Drilling

A continuous core sample can be obtained using a diamond tipped core barrel, usually with a 50 mm internal diameter. Provided full core recovery is achieved (which is not always possible in weak rocks and granular soils), this technique provides a very reliable method of investigation.

Standard Penetration Tests

Standard penetration tests (SPT) are used as a means of estimating the density or strength of soils and also of obtaining a relatively undisturbed sample. The test procedure is described in Australian Standard 1289, Methods of Testing Soils for Engineering Purposes - Test 6.3.1.

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 increments and the 'N' value is taken as the number of blows for the last 300 mm. 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.

- In the case where full penetration is obtained with successive blow counts for each 150 mm of, say, 4, 6 and 7 as:

  4,6,7
  N=13

- In the case where the test is discontinued before the full penetration depth, say after 15 blows for the first 150 mm and 30 blows for the next 40 mm as:

  15, 30/40 mm

The results of the SPT tests can be related empirically to the engineering properties of the soils.

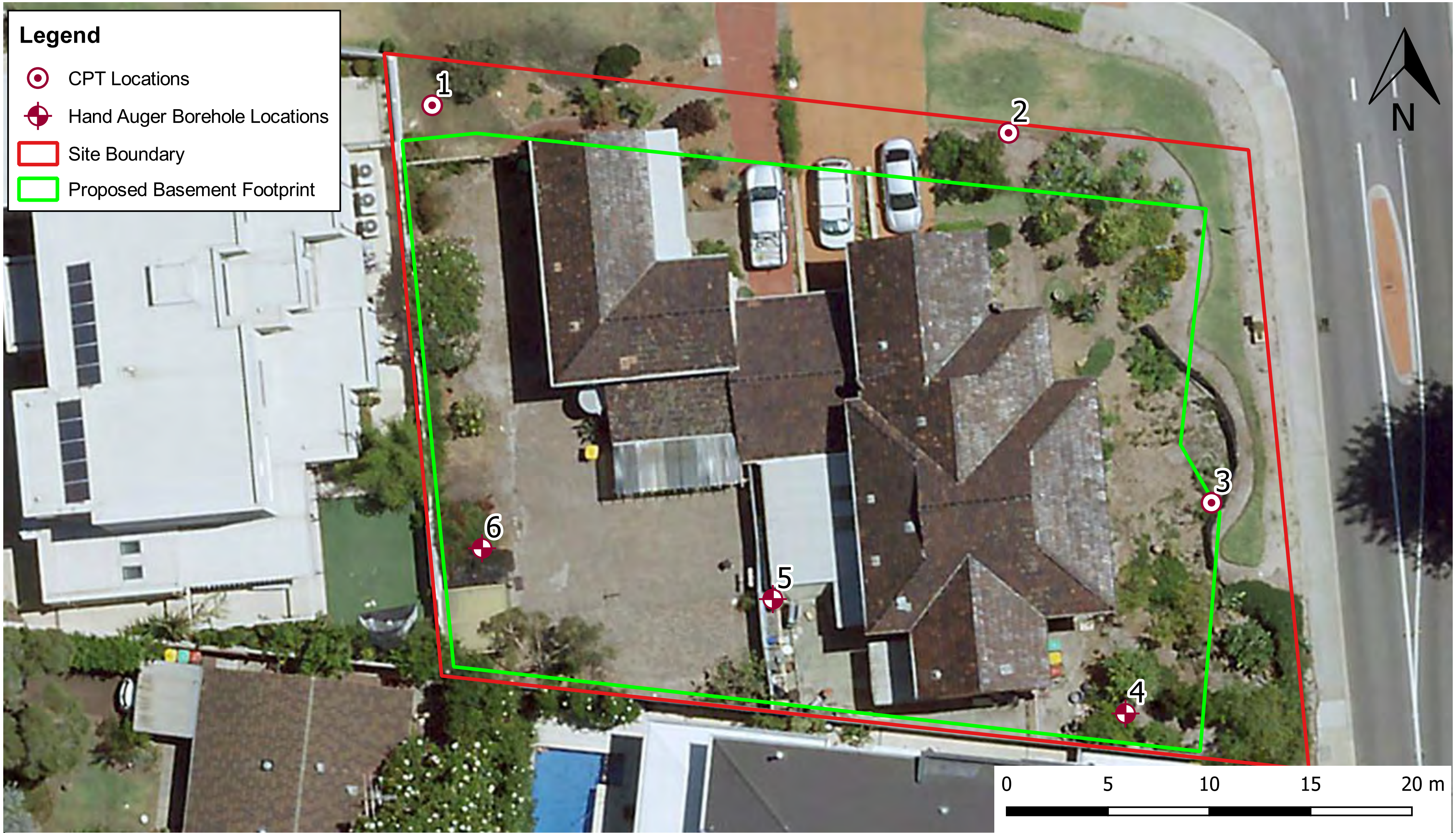
Dynamic Cone Penetrometer Tests / Perth Sand Penetrometer Tests

Dynamic penetrometer tests (DCP or PSP) are carried out by driving a steel rod into the ground using a standard weight of hammer falling a specified distance. As the rod penetrates the soil the number of blows required to penetrate each successive 150 mm depth are recorded. Normally there is a depth limitation of 1.2 m, but this may be extended in certain conditions by the use of extension rods. Two types of penetrometer are commonly used.

- Perth sand penetrometer - a 16 mm diameter flat ended rod is driven using a 9 kg hammer dropping 600 mm (AS 1289, Test 6.3.3). This test was developed for testing the density of sands and is mainly used in granular soils and filling.
- Cone penetrometer - a 16 mm diameter rod with a 20 mm diameter cone end is driven using a 9 kg hammer dropping 510 mm (AS 1289, Test 6.3.2). This test was developed initially for pavement subgrade investigations, and correlations of the test results with California Bearing Ratio have been published by various road authorities.

Appendix B

Drawing



Douglas Partners Geotechnics / Environment / Groundwater	**Test Locations** **Proposed Apartments Development** **20 Helm Street, Mount Pleasant, WA**	PROJECT:	201795.00
		Drawing No:	1
		REV:	0
	CLIENT: Pyramid Constructions (WA) Pty Ltd	DATE:	8/3/2021

Appendix C

Results of Field Work

CONE PENETRATION TEST

CLIENT: Pyramid Constructions (WA) Pty Ltd

PROJECT: Proposed Apartments Development

LOCATION: Mount Pleasant, WA

REDUCED LEVEL: RL 3.2 m

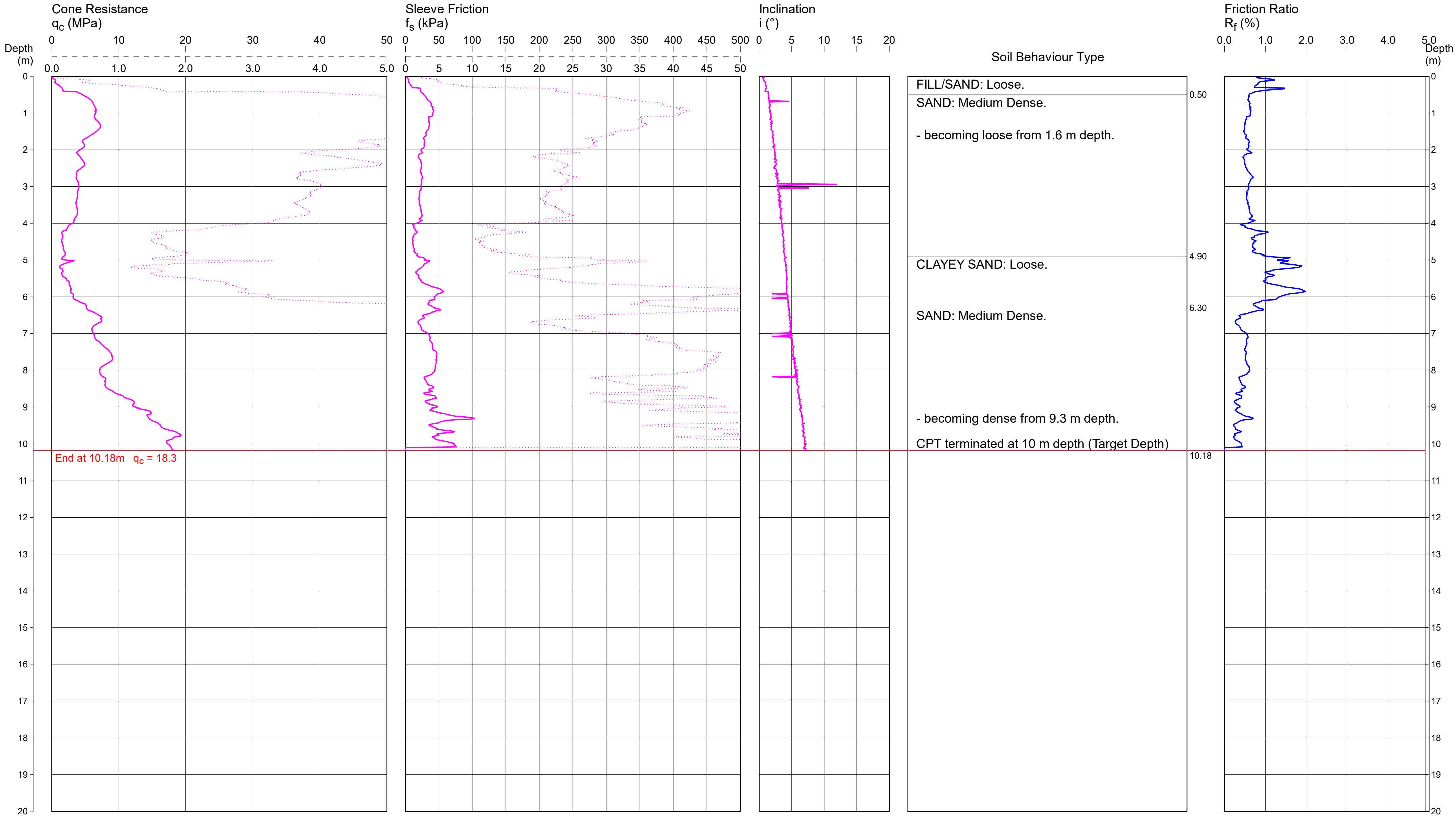
COORDINATES: 391426E 6457292N

1

Page 1 of 1

DATE 26/02/2021

PROJECT No: 201795.00



REMARKS: *Surface level inferred from publicly available LiDAR data.

File: P:\201795.00 - MOUNT PLEASANT, 20 Helm Street\4.0 Field Work\CPT\Douglas Partners\1.CP5
Cone ID: Probedrill **Type:** ECF21GM

ConePlot Version 5.9.2

Douglas Partners
Geotechnics | Environment | Groundwater

CONE PENETRATION TEST

CLIENT: Pyramid Constructions (WA) Pty Ltd

PROJECT: Proposed Apartments Development

LOCATION: Mount Pleasant, WA

REDUCED LEVEL: RL 2.6 m

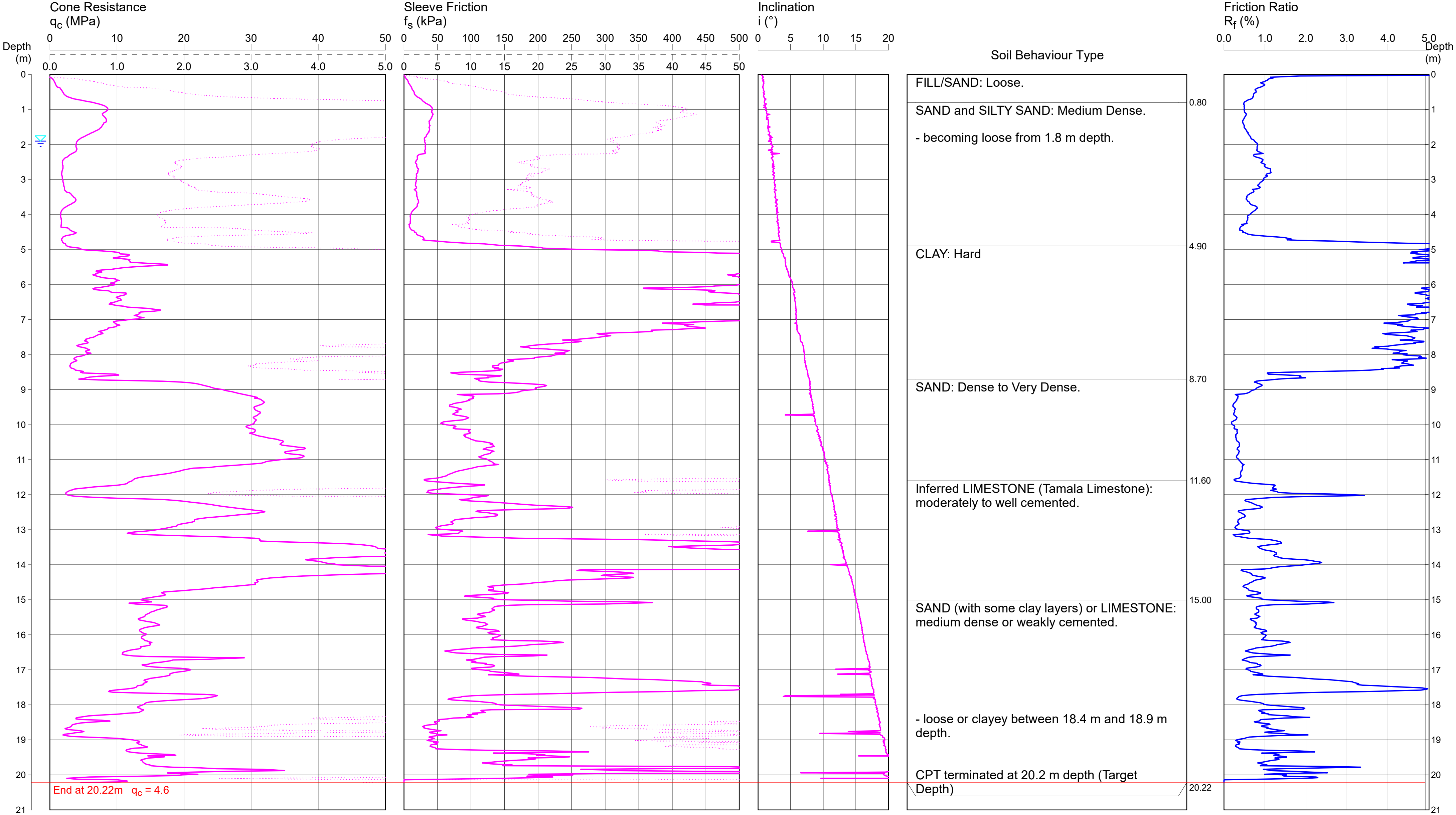
COORDINATES: 391454E 6457291N

2

Page 1 of 1

DATE 26/02/2021

PROJECT No: 201795.00



REMARKS: *Surface level inferred from publicly available LiDAR data.

File: P:\201795.00 - MOUNT PLEASANT, 20 Helm Street\4.0 Field Work\CPT\Douglas Partners\2.CP5

Cone ID: Probedrill **Type:** ECF21GM

ConePlot Version 5.9.2

Water depth after test: 1.90m depth (measured)

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CONE PENETRATION TEST

CLIENT: Pyramid Constructions (WA) Pty Ltd

PROJECT: Proposed Apartments Development

LOCATION: Mount Pleasant, WA

REDUCED LEVEL: RL 2.9 m

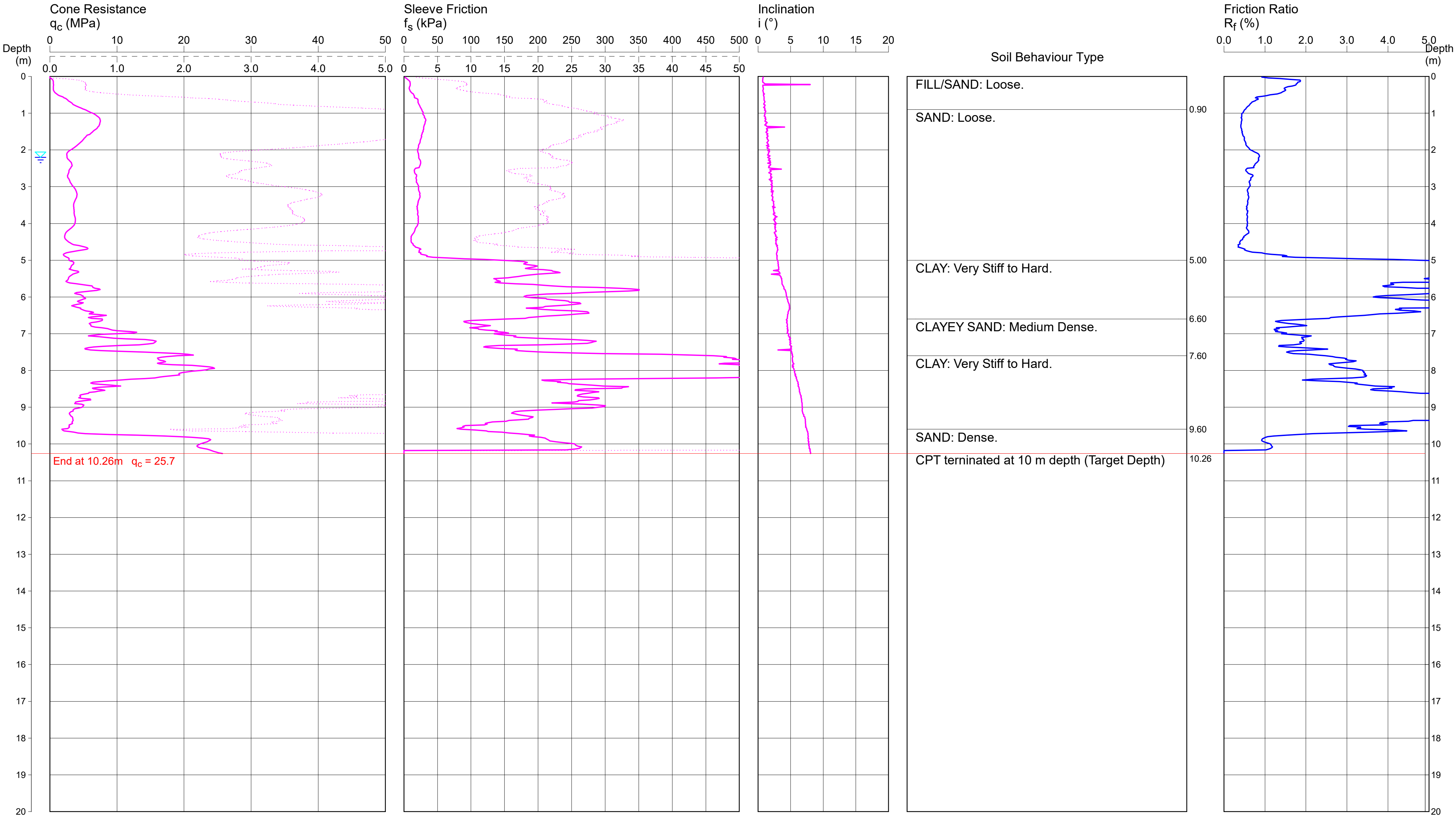
COORDINATES: 391464E 6457272N

3

Page 1 of 1

DATE 26/02/2021

PROJECT No: 201795.00



REMARKS: *Surface level inferred from publicly available LiDAR data.

File: P:\201795.00 - MOUNT PLEASANT, 20 Helm Street\4.0 Field Work\CPT\Douglas Partners\3.CP5

Cone ID: Probedrill **Type:** ECF21GM

ConePlot Version 5.9.2

Water depth after test: 2.20m depth (measured)

Douglas Partners
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BOREHOLE LOG

CLIENT: Pyramid Constructions (WA) Pty Ltd
PROJECT: Proposed Apartment Development
LOCATION: Mount Pleasant, WA

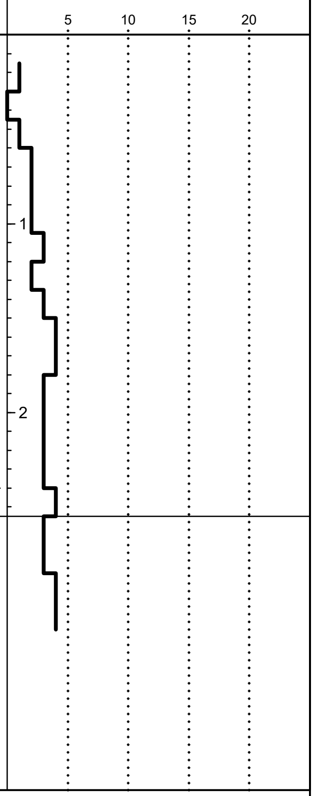
SURFACE LEVEL: 2.9 AHD
EASTING: 391460
NORTHING: 6457262
DIP/AZIMUTH: 90°/--

BORE No: 4
PROJECT No: 201795.00
DATE: 26/2/2021
SHEET 1 OF 1

RL	Depth (m)	Description of Strata	Graphic Log	Type	Depth	Sample	Results & Comments	Water
		FILL/SAND SP-SM: fine to medium grained, grey-brown, with silt, trace rootlets, moist, loose.		D	0.3			
	0.5	SAND SP: fine to medium grained, pale grey, trace silt, moist, loose. Eolian.		D	1.1			
		- becoming moist to wet from 2.0 m depth.						
		- becoming yellow-brown from 2.3 m depth.		D	2.5			▼ 26-02-21
	2.55	Bore discontinued at 2.55m (collapsing conditions)						

RL scale: 2, 1, 0, -1. Depth scale: 1, 2, 3.

Dynamic Penetrometer Test (blows per 150mm)



RIG: 110 mm hand auger **DRILLER:** PD **LOGGED:** PD **CASING:** None
TYPE OF BORING:
WATER OBSERVATIONS: Groundwater observed at approximately 2.4 m depth.
REMARKS: Location coordinates are in MGA94 Zone 50 J. Surface levels interpolated using LiDAR 5 m

☒ Sand Penetrometer AS1289.6.3.3
☐ Cone Penetrometer AS1289.6.3.2

SAMPLING & IN SITU TESTING LEGEND

A	Auger sample	G	Gas sample	PID	Photo ionisation detector (ppm)
B	Bulk sample	P	Piston sample	PL(A)	Point load axial test Is(50) (MPa)
BLK	Block sample	U_x	Tube sample (x mm dia.)	PL(D)	Point load diametral test Is(50) (MPa)
C	Core drilling	W	Water sample	pp	Pocket penetrometer (kPa)
D	Disturbed sample	▷	Water seep	S	Standard penetration test
E	Environmental sample	▼	Water level	V	Shear vane (kPa)

Douglas Partners
Geotechnics / Environment / Groundwater

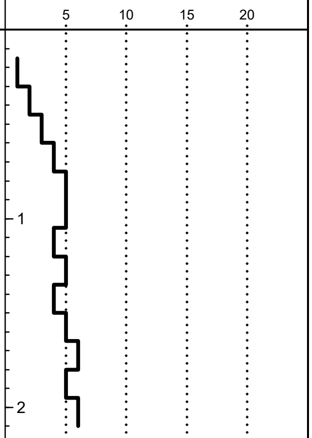
BOREHOLE LOG

CLIENT: Pyramid Constructions (WA) Pty Ltd
PROJECT: Proposed Apartment Development
LOCATION: Mount Pleasant, WA

SURFACE LEVEL: 2.7 AHD
EASTING: 391442
NORTHING: 6457266
DIP/AZIMUTH: 90°/--

BORE No: 5
PROJECT No: 201795.00
DATE: 26/2/2021
SHEET 1 OF 1

RL	Depth (m)	Description of Strata	Graphic Log	Type	Depth	Sample	Results & Comments	Water	Dynamic Penetrometer Test (blows per 150mm)
		FILL/SAND SP-SM: fine to medium grained, grey-brown, with silt, trace rootlets, dry to moist, loose. - becoming medium dense from 0.45 m depth.							
	0.85	SAND SP: fine to medium grained, pale grey, trace silt, moist, medium dense. Eolian.		D	1.2				
	1.6	SAND SP-SM: fine to medium grained, brown mottled orange-brown, with silt, moist to wet, medium dense, weakly cemented (coffee rock). - becoming pale brown from 1.85 m depth. - becoming wet and pale grey from 2.4 m depth. - becoming yellow-brown from 2.5 m depth.		D	2.0			26-02-21	
	2.55	Bore discontinued at 2.55m (collapsing conditions)							



RIG: 110 mm hand auger **DRILLER:** PD **LOGGED:** PD **CASING:** None
TYPE OF BORING:
WATER OBSERVATIONS: Groundwater observed at approximately 2.4 m depth.
REMARKS: Location coordinates are in MGA94 Zone 50 J. Surface levels interpolated using LiDAR 5 m

☒ Sand Penetrometer AS1289.6.3.3
☐ Cone Penetrometer AS1289.6.3.2

SAMPLING & IN SITU TESTING LEGEND

A	Auger sample	G	Gas sample	PID	Photo ionisation detector (ppm)
B	Bulk sample	P	Piston sample	PL(A)	Point load axial test Is(50) (MPa)
BLK	Block sample	U_x	Tube sample (x mm dia.)	PL(D)	Point load diametral test Is(50) (MPa)
C	Core drilling	W	Water sample	pp	Pocket penetrometer (kPa)
D	Disturbed sample	▷	Water seep	S	Standard penetration test
E	Environmental sample	▼	Water level	V	Shear vane (kPa)

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Geotechnics | Environment | Groundwater

BOREHOLE LOG

CLIENT: Pyramid Constructions (WA) Pty Ltd
PROJECT: Proposed Apartment Development
LOCATION: Mount Pleasant, WA

SURFACE LEVEL: 2.8 AHD
EASTING: 391428
NORTHING: 6457273
DIP/AZIMUTH: 90°/--

BORE No: 6
PROJECT No: 201795.00
DATE: 26/2/2021
SHEET 1 OF 1

RL	Depth (m)	Description of Strata	Graphic Log	Sampling & In Situ Testing: Type	Depth	Sample	Results & Comments	Water	Dynamic Penetrometer Test (blows per 150mm)
		FILL/SAND SP-SM: fine to medium grained, grey-brown, with silt, trace rootlets, moist, loose.		D	0.7				
		- becoming medium dense from 0.75 m depth.							
	1.05	SAND SP: fine to medium grained, pale grey, trace silt, moist, loose to medium dense. Eolian.							
		- becoming brown from 1.9 m depth.							
		- becoming moist to wet from 2.0 m depth.							
		- becoming orange-brown and wet from 2.3 m depth.		D	2.4			26-02-21	
	2.6	Bore discontinued at 2.6m (collapsing conditons)							

RIG: 110 mm hand auger **DRILLER:** PD **LOGGED:** PD **CASING:** None
TYPE OF BORING:
WATER OBSERVATIONS: Groundwater observed at approximately 2.45 m depth.
REMARKS: Location coordinates are in MGA94 Zone 50 J. Surface levels interpolated using LiDAR 5 m

☒ Sand Penetrometer AS1289.6.3.3
☐ Cone Penetrometer AS1289.6.3.2

SAMPLING & IN SITU TESTING LEGEND

A	Auger sample	G	Gas sample	PID	Photo ionisation detector (ppm)
B	Bulk sample	P	Piston sample	PL(A)	Point load axial test Is(50) (MPa)
BLK	Block sample	U_x	Tube sample (x mm dia.)	PL(D)	Point load diametral test Is(50) (MPa)
C	Core drilling	W	Water sample	pp	Pocket penetrometer (kPa)
D	Disturbed sample	▷	Water seep	S	Standard penetration test
E	Environmental sample	▼	Water level	V	Shear vane (kPa)

Douglas Partners
Geotechnics / Environment / Groundwater

Appendix D

Laboratory Test Results

WESTERN GEOTECHNICAL & Laboratory Services

SOIL | AGGREGATE | CONCRETE | CRUSHING

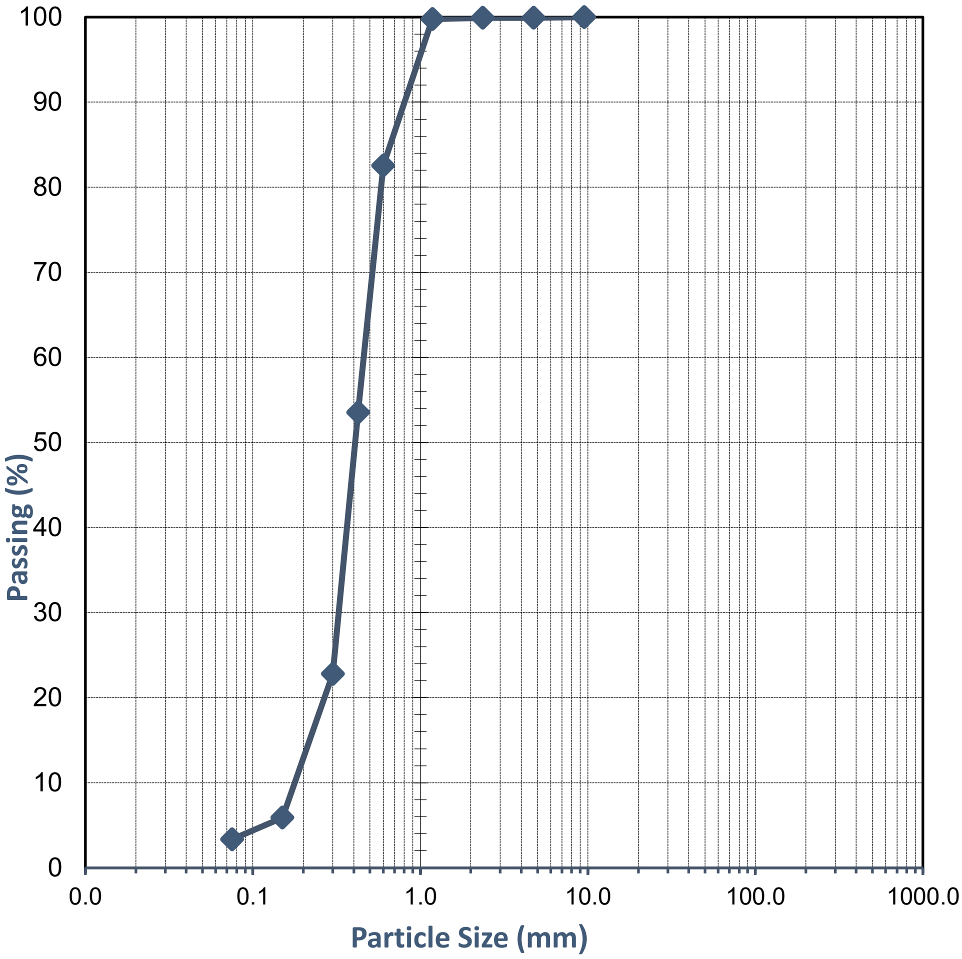
TEST REPORT - AS 1289.3.6.1

Client:	Pyramid Constructions (WA) Pty Ltd	*Ticket No.*	S2606
Client Address:	-	*Report No.*	WG21/3358_1_PSD
Project:	Proposed Apartment Development	*Sample No.*	WG21/3358
Location:	Mount Pleasant, WA	*Date Sampled:*	26/02/2021
Sample Identification:	4, 2.5m	*Date Tested:*	4/3 - 5/3/2021

TEST RESULTS - Particle Size Distribution of Soil

Sampling Method: **Sampled by Client, Tested as Received**

Sieve Size (mm)	Percent Passing Sieve (%)
150.0	
100.0	
75.0	
37.5	
19.0	
9.5	100
4.75	100
2.36	100
1.18	100
0.600	83
0.425	54
0.300	23
0.150	6
0.075	3



Comments:

Approved Signatory:

Name: Brooke Elliott

Date: 05/March/2021

NATA WORLD RECOGNISED ACCREDITATION

Accreditation No. 20599
Accredited for compliance
with ISO/IEC 17025 - Testing

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WESTERN GEOTECHNICAL & Laboratory Services

SOIL | AGGREGATE | CONCRETE | CRUSHING

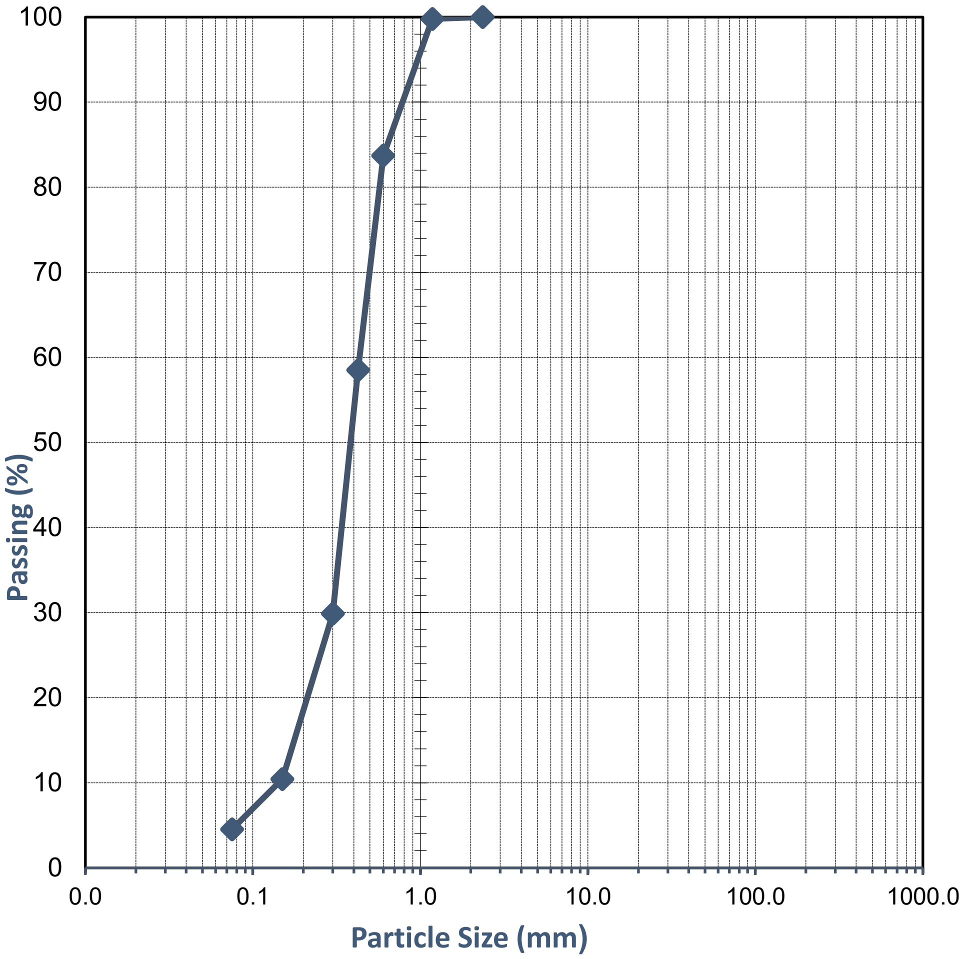
TEST REPORT - AS 1289.3.6.1

Client:	Pyramid Constructions (WA) Pty Ltd	*Ticket No.*	S2606
Client Address:	-	*Report No.*	WG21/3359_1_PSD
Project:	Proposed Apartment Development	*Sample No.*	WG21/3359
Location:	Mount Pleasant, WA	*Date Sampled:*	26/02/2021
Sample Identification:	6, 2.4m	*Date Tested:*	4/3 - 5/3/2021

TEST RESULTS - Particle Size Distribution of Soil

Sampling Method: **Sampled by Client, Tested as Received**

Sieve Size (mm)	Percent Passing Sieve (%)
150.0	
100.0	
75.0	
37.5	
19.0	
9.5	
4.75	
2.36	100
1.18	100
0.600	84
0.425	59
0.300	30
0.150	10
0.075	5



Comments:

Approved Signatory:

Name: Brooke Elliott

Date: 05/March/2021

Accreditation No. 20599
Accredited for compliance
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Envirolab Services (WA) Pty Ltd trading as MPL Laboratories
ABN 53 140 099 207
16-18 Hayden Court Myaree WA 6154
ph 08 9317 2505   fax 08 9317 4163
lab@mpl.com.au
www.mpl.com.au

CERTIFICATE OF ANALYSIS 258093

Client Details

Client	Western Geotechnical & Laboratory Services
Attention	Brooke Elliott
Address	235 Bank Street, Welshpool, WA, 6101

Sample Details

Your Reference	**S2606 - Pyramid Constructions (WA)**
Number of Samples	2 Soil
Date samples received	04/03/2021
Date completed instructions received	04/03/2021

Analysis Details

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

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

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

Report Details

Date results requested by	11/03/2021
Date of Issue	11/03/2021

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Results Approved By
Heram Halim, Operations Manager

Authorised By

Michael Kubiak, Laboratory Manager

Miscellaneous Inorg - soil			
Our Reference		258093-1	258093-2
Your Reference	UNITS	WG21/3357 - 4, 0.3m	WG21/3359 - 6, 2.4m
Depth		0.3	2.4
Date Sampled		26/02/2021	26/02/2021
Type of sample		Soil	Soil
Date prepared	-	08/03/2021	08/03/2021
Date analysed	-	08/03/2021	08/03/2021
pH	pH Units	7.6	7.7
Sulphate	mg/kg	38	<10
Chloride	mg/kg	64	16

Method ID	Methodology Summary
INORG-001	pH - Measured using pH meter and electrode base on APHA latest edition, Method 4500-H+. Please note that the results for water analyses may be indicative only, as analysis can be completed outside of the APHA recommended holding times. Soils are reported from a 1:5 water extract unless otherwise specified.
INORG-081	Anions - a range of anions are determined by Ion Chromatography based on APHA latest edition Method 4110-B. Soils and other sample types reported from a water extract unless otherwise specified (standard soil extract ratio 1:5).

QUALITY CONTROL: Miscellaneous Inorg - soil					Duplicate				Spike Recovery %	
Test Description	Units	PQL	Method	Blank	#	Base	Dup.	RPD	LCS-1	[NT]
Date prepared	-			08/03/2021	[NT]	[NT]	[NT]	[NT]	08/03/2021	[NT]
Date analysed	-			08/03/2021	[NT]	[NT]	[NT]	[NT]	08/03/2021	[NT]
pH	pH Units		INORG-001	[NT]	[NT]	[NT]	[NT]	[NT]	101	[NT]
Sulphate	mg/kg	10	INORG-081	<10	[NT]	[NT]	[NT]	[NT]	97	[NT]
Chloride	mg/kg	10	INORG-081	<10	[NT]	[NT]	[NT]	[NT]	94	[NT]

Result Definitions

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

Quality Control Definitions

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

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

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

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

Laboratory Acceptance Criteria

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

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

Spikes for Physical and Aggregate Tests are not applicable.

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

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

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

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

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

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

Measurement Uncertainty estimates are available for most tests upon request.

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.