Douglas Partners
Geotechnics | Environment | Groundwater

Report on
Geotechnical Investigation

Proposed Ergon Depot
Cnr Sheridan and Hartley Streets, Cairns

Prepared for
Clarke & Prince Pty Ltd Architects

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Report on Geotechnical Investigation
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cnr Sheridan and Hartley Streets, Cairns

1. Introduction

This report presents the results of a geotechnical investigation carried out for a proposed Ergon depot, to be built at the corner of Sheridan and Hartley Streets, Cairns. The investigation was commissioned in an email dated 28 July 2022 from Mr Martin Majer of Clarke and Prince Pty Ltd (project Architects), and was undertaken in accordance with Douglas Partners Pty Ltd (DP) proposal 216856.00.P.001.Rev0 dated 26 July 2022.

It is understood that the proposed development will comprise the construction of a single storey building with a suspended floor slab, located in the southern corner of the site.

The purpose of the investigation was to assess subsurface conditions at the site, and provide engineering comment on the following:

- results of geotechnical and ASS laboratory testing;
- site preparation earthworks;
- site classification to AS 2870 (2011);
- suitability of high-level footings;
- settlement due to fill placement, and of high-level footings, based on supplied loadings;
- comments on suitable pile types, and end bearing and skin friction design parameters for piles loaded vertically in compression;
- calculated ϕ_{gb} limit state reduction factor, based on AS2159 (2009);
- interpretive commented on the results of the preliminary acid sulfate laboratory testing, and the presence or otherwise of Actual Acid Sulfate Soils (AASS) or Potential Acid Sulfate Soils (PASS) at the tested locations;
- preliminary lime neutralisation rates (if AASS or PASS is encountered); and
- recommendations as to whether or not an Acid Sulfate Soils Management Plan (ASSMP) is required.

The scope of work for the investigation comprised two cone penetration tests (CPTs), and the excavation of three test pits, followed by laboratory testing, engineering evaluation, analysis and reporting.

This report must be read in conjunction with the notes About This Report in Appendix A along with any other attached explanatory notes and should be kept in its entirety without separation of individual pages or sections.

2. Site Description

The site is located in the southern corner of a vacant allotment located at the northern side of the intersection of Sheridan and Hartley Streets, Cairns, as shown on Drawing 1 in Appendix B.

A photograph of the site taken during the field work, is shown below as Figure 1.

The rectangular shaped site measures approximately 40 m along the south eastern Hartley Street boundary, and 30 m along the south western Sheridan Street boundary. The site is bound to the north west by an Ergon substation building, and to the north east by a vacant allotment being used as a storage area for building materials.

At the time of the field work, the site was flat and vacant, and partly vegetated by short grass, with some garden beds with established trees partially along both road frontages. Several low piles of spoil (reportedly ‘sucker’ truck discharge) were located in the northern portion of the site, beyond the proposed location for the depot.

Figure 1: View of the site looking south west towards the corner of Sheridan and Hartley Streets.

3. Geology

Reference to the 1:100 000 Queensland Department of Resources detailed surface geology (2018), and accompanying notes indicates that the site is underlain by Holocene aged alluvial deposits, typically described as “silt, mud, sand and minor salt; coastal tidal flats, mangrove flats, supratidal flats, saltpans and grasslands.

The subsurface conditions encountered at the test locations comprised fill, over soft silty clay and organic silty clay. Underlying this, more consolidated clays and sands were encountered at depth These results are considered consistent with previous experience, and the above described geological mapping.

4. Field Work Methods

The field work for this investigation was undertaken on 4 August 2022 and comprised the excavation of three test pits (designated Pits 1 to 3) to between 2.0 m and 2.4 m depth, and two CPTs (designated CPT 1 and 3 and undertaken adjacent to the pit of the same number), to depths of around 9 m. The test locations are shown on Drawing 1 in Appendix B.

In the CPT, a 35 mm diameter cone and 135 mm long friction sleeve were advanced into the ground at a constant rate of 20 mm per second, by hydraulic thrust from a purpose-built portable test rig temporarily ballasted by a 10 t excavator. Cone and friction sleeve readings were measured continuously with depth, and captured at 20 mm intervals on a computer for plotting and processing. Additional information on the CPT method and interpretation is given in the notes in Appendix A.

The test pits were excavated using a 10 tonne excavator with a 450 mm wide blade bucket. Representative samples were collected from the bucket at regular depth intervals in each pit for both geotechnical and environmental laboratory testing, and visual and tactile classification purposes. Disturbed samples were also generally collected at 0.25 m depth intervals in the pits for the purposes of ASS testing. In-situ pocket penetrometer (pp) tests were carried out on cohesive soils (silts and clays) to assess the soil strength consistency. A dynamic cone penetrometer (DCP) test was attempted adjacent to Pits 1 and 3, however, both encountered refusal at shallow depth.

The field work was carried out under the direction of a geotechnical engineer who: supervised the CPTs and recorded the results using Cone Plot software; logged the soil profile in the test pits; recorded groundwater levels; collected representative samples for laboratory testing and classification purposes; and carried out the DCPs.

The existing ground surface level and UTM co-ordinates at each test location, to AHD and GDA datums, was determined using a dGPS unit accurate to approximately 0.1 m.

5. Field Work Results

The subsurface conditions encountered in the test pits, and inferred from the CPTs, are presented in Appendix C. These should be read in conjunction with the general notes in Appendix A which explain descriptive terms and classification methods used in their preparation.

The subsurface conditions generally comprise fill over Holocene muds, then silty clay and consolidated alluvials which are further described below:

Fill Encountered/inferred at each test location, to depths of 0.65 m or 0.7 m, and comprised very dense granular material (silty sand, clayey gravelly sand, silty clayey gravel, sand and sandy gravel) to 0.25 m or 0.5 m depth, over hard cohesive material (clayey silt, gravelly clayey silt, or sandy silty clay).

In the absence of ‘Level 1’ inspection and testing records as detailed in AS 3798:2007, this fill is considered ‘uncontrolled’, despite the apparent relatively good compaction observed.

Holocene Mud	Natural silty clay, organic silty clay, or sandy silty clay was encountered/inferred below the fill, and continued to test pit termination (at 2 m and 2.4 m depth) in Pits 1 to 3, and 6 m and 6.15 m depth in CPT 1 and 3, respectively. These deposits were initially stiff, grading firm, then soft below 1.6 m depth. In CPT 1, this unit graded firm below 5 m depth. In Pits 1 to 3, this unit comprised silty clay upon contact, then organic silty clay near the base of each pit.
Silty Clay	Inferred below the Holocene Mud in CPTs 1 and 3, and continued to around 8.3 m depth; initially stiff then very stiff below around 6.9 m depth.
Consolidated Alluvials	Below 8.15 m depth in CPT 1 and 8.3 m depth in CPT 3, and comprised sands and clayey sand of medium dense to dense relative density interbedded with hard clay, and inferred to CPT termination at approximately 9 m depth.

Localised groundwater seepages were observed into the test pits during excavation at below 1.7 m depth in Pits 1 and 2, and below 1.6 m depth in Pit 3. It should be noted that groundwater levels are affected by climatic conditions and by soil permeability, and will therefore vary with time. Furthermore, Cairns is located within the wet tropics and is particularly susceptible to 'wet' and 'dry' seasonal variations in rainfall.

6. Laboratory Testing

6.1 Atterberg Limit / Linear Shrinkage Tests

Laboratory testing comprised two Atterberg limit / linear shrinkage tests. The laboratory test results are attached in Appendix D, and summarised in Table 1 below.

Table 1 – Summary of Laboratory Test Results

Pit	Depth (m)	Material	FMC (%)	LL (%)	PL (%)	PI (%)	LS (%)
1	0.6	FILL - clayey SILT	9.9	29	24	5	1.0
2	1.0	Silty CLAY	33.7	60	21	39	15.5

Legend:
FMC = field moisture content
LL = liquid limit
PI = plasticity index
PL = plasticity limit
LS = linear shrinkage

6.2 Acid Sulfate Screening and Chromium Suite Tests

Field screening and laboratory testing for ASS was carried out with reference to the QASSIT Guidelines (1998), the Soil Management Guidelines (2014) and the Laboratory Methods Guidelines (2004).

Samples collected from the pits were screened by measurement of pH after the addition of distilled water (pH_F) and peroxide (pH_{FOX}). This was in order to give an approximate indication of either the presence of AASS or PASS conditions.

Based on the results of the screening tests, eight samples were subjected to detailed analysis using the Chromium Suite of tests conducted by SGS Australia Pty Ltd, a NATA registered analytical laboratory. The results of the Chromium Suite laboratory tests are provided in Appendix D and are also summarised in Table 2 below, along with the results of the screening tests.

Table 2 - Results of ASS Field Screening and Chemical Laboratory Testing

Pit	Depth (m)	Sample Description	Screening Test Results				Chromium Suite Test Results (% w/w S)				
			pH_F	pH_{FOX}	Δ pH	Reaction (1,2,3,4)* F**	pH_{KCL}	Potential Sulfidic Acidity (S_{CR})	Titratable Actual Acidity (s-TAA)	ANC (s-ANCBT)	Existing + Potential Acidity
1	0.25	FILL: Silty sand	7.7	7.6	0	1	-	-	-	-	-
	0.45	FILL: Silty clayey gravel	7.5	9.2	-1.7	4	-	-	-	-	-
	0.6	FILL: Clayey silt	8.0	8.8	-0.8	4	8.2	<0.005	<0.01	0.04	<0.005
	0.75	Silty clay	8.4	8.4	0	F	-	-	-	-	-
	1.0	Silty clay	8.2	7.2	1.0	F	-	-	-	-	-
	1.25	Silty clay	8.5	7.2	1.3	F	8.5	0.035	<0.01	2.9	0.035
	1.5	Silty clay	8.4	7.2	1.2	F	-	-	-	-	-
	1.75	Silty clay	7.9	6.0	1.9	F	-	-	-	-	-
	2.0	Organic silty clay	7.6	1.0	6.6	1	6.8	1.7	<0.01	1.4	1.7
2	0.25	FILL: Clayey gravelly sand	8.8	6.7	2.1	3	-	-	-	-	-
	0.45	FILL: Clayey gravelly sand	8.7	8.4	0.3	2	9.1	<0.005	<0.01	0.69	<0.005
	0.6	FILL: Gravelly clayey silt	8.3	8.6	-0.3	1	-	-	-	-	-
	0.75	Silty clay	7.9	8.3	-0.4	4	-	-	-	-	-
	1.0	Silty clay	7.9	7.6	0.3	4	-	-	-	-	-
	1.25	Silty clay	8.0	7.5	0.5	4	-	-	-	-	-
	1.5	Silty clay	7.6	5.8	1.8	F	7.4	0.007	<0.01	0.12	0.007
	1.75	Organic silty clay	7.2	1.0	6.2	3	-	-	-	-	-
	2.0	Organic silty clay	7.4	1.4	6.0	4	-	-	-	-	-
3	0.2	FILL: Sandy gravel	8.4	7.4	1.0	2	-	-	-	-	-
	0.3	FILL: Sandy silty clay	8.2	6.7	1.5	F	8.7	<0.005	<0.01	1.4	<0.005
	0.5	FILL: Gravelly clayey silt	8.1	7.2	0.9	F	-	-	-	-	-
	0.75	Silty clay	7.9	7.8	0.1	3	-	-	-	-	-
	1.0	Silty clay	8.1	8.0	0.1	4	8.4	<0.005	<0.01	2.5	<0.005
	1.25	Silty clay	8.1	8.0	0.1	4	-	-	-	-	-
	1.5	Silty clay	7.9	7.3	0.6	4	-	-	-	-	-
	1.75	Organic silty clay	7.7	0.7	7.0	F	6.8	1.3	<0.01	0.53	1.3
	2.0	Organic silty clay	7.5	0.9	6.6	F	-	-	-	-	-

Notes to Table 2:

* 1 – denotes slight effervescence: 2 – denotes moderate reaction: 3 – denotes vigorous reaction:
4 – denotes very strong effervescence accompanied by escape of gas/heat

** F – indicates a bubbly/froth reaction (organics) ND – not determined

7. Comments

7.1 Proposed Development

It is understood that the proposed development will comprise the construction of a single storey building with finished floor level at RL 3.45 mAHD, located in the southern corner of the site. Based on the supplied survey plan, existing site levels at the subject site are of the order of RL1.9 m to RL2 m, resulting in the proposed floor level being approximately 1.5 m above existing site levels.

Information on the structural loading of the building was not available at the time of reporting. Based on discussions with the client, it is anticipated that the building would be supported on a piled footing system, with a un-filled void beneath the floor. For completeness, an estimate of settlement due to load from placement of 1.4 m depth of fill, as well as the building load supported on the fill, are presented below, and on the assumption that these settlements preclude further consideration of high level footings, no further comment regarding high level footings is provided.

In addition, it is understood that the remainder of the site would be developed as a carpark, at approximately the current elevation.

7.2 Appreciation of Subsurface Conditions

The results of the investigation indicate the presence of relatively poor subsurface conditions. Significant geotechnical issues relating to the proposed development at the site include the following:

- the existing fill, of 0.65 m to 0.7 m thickness, which although typically either very dense or hard at the test locations is ‘uncontrolled’ and hence may vary in composition and compaction. There are associated risks of differential settlement for any structures relying on this fill for support. It may be suitable to be left in-situ and utilised as a bridging layer over the deeper and ‘poorer’ natural materials, provided the risks outlined above are accepted by the client;
- a 3.4 m to 4.4 m thick layer of Holocene mud below around 1.6 m depth, which:
    - is generally soft and will affect support for high-level footings;
    - is relatively compressible and will induce total and differential settlement under load; and
    - is known to be a relatively strong PASS and will require appropriate environmental management during earthworks if disturbed.

These issues are further discussed in the following sections of this report.

7.3 Site Classification to AS 2870

The site classification in accordance with AS 2870 (2011) is 'Class P', requiring design by engineering principles. The reasons for this include likely excessive settlement resulting from building and fill loads, and the presence of 'uncontrolled' fill.

Footing design for the proposed building should therefore be undertaken by engineering principles.

7.4 Site Preparation

If the existing fill is to be left in-situ, and utilised as a subgrade for carpark pavements, then it is suggested that additional geotechnical assessment of the existing fill be undertaken, following the stripping of topsoil, as outlined below. Alternatively, the existing fill could be removed and replaced as controlled fill, although placement of a granular bridging layer on the surface of the natural silty clay exposed by removal of the fill may be required, and the requirements for this should be assessed by geotechnical inspection at the time of construction if the existing fill is to be removed.

Site preparation should include the following:

- Remove all existing surface vegetation, organic topsoil, and any deleterious soft, wet or highly compressible materials, as well as any existing footings, structures or in-ground services.
- Undertake additional geotechnical investigation comprising DCPs through the existing fill, under the inspection of an experienced geotechnical engineer, on a nominal 5 m grid across the proposed pavement footprint.
- Undertake review of the additional DCP test results by an experienced geotechnical engineer. If the results indicate poorer/softer conditions than those indicated at the existing test locations, then it is likely that the existing fill will need to be removed.
- Test roll the exposed surface of the 'uncontrolled' fill (or natural ground beneath if the fill is removed) with at least six passes of a minimum 12 t static weight smooth drum roller (in non-vibratory mode). The final test roll pass should be accompanied by careful inspection by a geotechnical engineer to detect any remaining relatively soft or loose zones within the 'uncontrolled' fill, which should be excavated out and replaced with approved engineered fill under 'Level 1' inspection and testing;
- Place approved engineered fill in layers not exceeding 200 mm loose thickness, and compact to at least 98% standard dry density ratio or density index of at least 75%. Moisture contents within cohesive fill should be maintained within 2% of standard optimum moisture content, during and after compaction; and
- Undertake 'Level 1' inspection and testing for all additional fill placement works, in accordance with (AS 3798, 2007).

The above procedures will require geotechnical inspection and testing services to be employed during construction.

In the footprint of the proposed piled building, minimal site preparation would be required, and should comprise a proof roll to identify and treat any obviously soft or loose zones which may result in poor trafficability during construction. A specific platform thickness design would need to be undertaken for the support of any cranes and/or piling equipment. Geotechnical advice should be sought at the time of construction in this regard.

Further geotechnical advice should sought if it is proposed to fill the site to facilitate support of the building on a fill pad, to assess the risk of instability around the perimeter of the fill pad, due to the shallow underlying soft Holocene mud.

7.5 Settlements

An in-house spreadsheet, T-REX, which models fully-flexible rectangular areas loaded with uniform pressures, was used to estimate the settlement resulting from placement of 1.4 m thickness of additional fill (modelled as a uniform pressure of 28 kPa), as well as a uniform building slab pressure of 15 kPa. The ground conditions inferred in the CPTs were used to model the soil profile.

The estimated settlements at the centre of the loaded areas are presented in Table 3. These values are for fully-flexible load application. The settlement of a fully-rigid pad or strip footing is approximately 0.8 times the centre settlement but more evenly distributed. The behaviour of a raft slab footings is expected to lie somewhere between fully-flexible and fully-rigid conditions, depending upon slab rigidity.

In addition to the elastic settlements, long term creep settlements will occur within the soft Holocene muds below large loaded areas (i.e. slabs and site fill). The phenomenon of creep is not well understood, and a variety of methods are available for its analysis. The additional creep settlements at 20 years after construction are very approximate, and presented in Table 3, and would act in addition to the estimated elastic settlement.

Table 3: Summary of Estimated Settlements of Fully-Flexible Loaded Areas

Loading	Uniform Applied Pressure (kPa)	Size of Loaded Area	Elastic Settlement (mm)		Creep at 20 years below Centre (mm)	Total Centre Settlement at 20 years (mm)
			Centre	Corner		
Raft Slab	15	12 m by 19 m	40 to 50	5 to 15	-	-
1.4 m Fill	28	12 m by 19 m	70 to 90	10 to 30	-	-
Raft Slab and 1.4 m Fill	as above		110 to 140	15 to 45	60 to 80	170 to 220

It should be noted that the soft Holocene muds may still be consolidating under the weight of the existing thickness of ‘uncontrolled’ fill at the site (up to 0.7 m deep). The time that this fill was placed is not known with any certainty, however, it is considered likely to have been placed many years ago. In this case, the majority of primary consolidation settlement due to the weight of this fill is likely to be complete, however some minor creep settlement may be still ongoing, as this continues indefinitely on a log time scale, and would be difficult to quantify.

7.6 Pile Design

7.6.1 General

Pile types that are suitable for use on this site include bottom driven cast-in-situ (‘Franki’) piles (or similar), driven precast concrete or treated timber piles, or steel screw piles. Bored piers are not likely to be suitable due to the presence of soft clay, which is likely to require the use of casing to avoid slumping of material into the unsupported excavation. The soft clay is also ASS (refer Section 7.8), and therefore would require appropriate environmental management of generated spoil. Vibrations from driven piles (precast, timber or ‘Franki’) would need to be carefully managed to minimise possible cracking and damage to any nearby buildings or structures with brittle components.

Comments are provided below on appropriate unfactored ultimate loadings for driven concrete or timber piles, as well as steel screw piles, which are the anticipated most suitable piles types for the single storey loads.

It is suggested that a factor of safety of 2.5 be applied to ultimate pile values for working stress analysis. Alternatively, after assessing the overall design average risk rating (ARR), in accordance with the guidelines presented in AS 2159 (2009), a geotechnical strength reduction factor of 0.45 is suggested for limit state design for the site, design and installation risk factors anticipated, for a low redundancy system. A higher geotechnical strength reduction factor may be applied if piles are subjected to load testing. This higher value will depend upon the percentage of piles to be tested, and the value of an intrinsic test factor which varies in accordance with the test routine adopted.

7.6.2 Driven Piles

Static analysis has been used to determine the ultimate pile capacities indicated below for driven timber or precast piles. It is recommended, however, that pile capacities also be checked in the field by the use of a suitable dynamic formula (such as Hiley or similar) applied to measured set and temporary compression, or more sophisticated dynamic testing methods, such as CAPWAP or PDA.

Driven piles founded in the medium dense or hard consolidated alluvials, as encountered below depths of about 8 m in CPTs 1 and 3, and founded a minimum of four pile diameters penetration into this founding material, may be designed for the following:

- 2500 kPa ultimate unfactored end bearing;
- 30 kPa ultimate unfactored skin friction, for that portion of the pile penetrating into the above described founding material.

7.6.3 Steel Screw Piles

Static analysis has been used to determine the bearing pressures indicated below for steel screw piles. It is recommended that the installation of steel screw piles should only be carried out by experienced contractors and that pile capacities should also be checked in the field by load testing after installation is complete. Measurement of installation torque should not be relied upon to indicate pile capacity, as it has been documented that this can lead to misleading results.

Structural capacity of steel screw piles should be checked, and due allowance made for inclined or eccentric loads, and possible corrosion effects.

In order to minimise deflection under test loads, helix plate thickness should be at least 40mm (or 25mm if pile working loads are less than about 600kN). For helix outstand to plate thickness ratios of greater than 10, consideration should be given to the possible formation of a plastic hinge, which may in effect reduce the effective diameter of the helix, and the resulting test load able to be resisted by the pile.

Furthermore, it should be noted that the geotechnical strength reduction factor (Φ_g) is typically 0.5, whereas the structural strength reduction factor (Φ_s) is typically 0.9. The results of steel screw pile load tests, however, typically indicate that plastic deformation of the helix can occur when a screw pile is test loaded to E_d/ϕ_g, ie twice the design action effect (E_d) for a Φ_g of 0.5, as required by AS 2159:2009. Structural failure would not be expected to occur at normal serviceability loads however.

For founding in the medium dense or hard consolidated alluvials, as encountered below depths of about 8 m in CPTs 1 and 3, and after penetration of at least one to two helix diameters into this unit, steel screw piles may be designed for an ultimate unfactored end bearing pressure of 1800 kPa.

It is usual practice not to allow for skin friction in the design of screw piles.

It should be noted that the alluvial soil profile present at the site, and across most of the Cairns CBD, is present to considerable depth (likely to be many tens of metres, possibly up to 100 m depth or more). Being an alluvial profile, the profile strength does not necessarily improve with depth, as is common with a residual ground profile (ie derived from weathered bedrock), as might be present at other CBD locations in Queensland where screw pile contractors might be based. Founding depths for screw piles should therefore be carefully assessed based on the results of this (and possibly additional) investigation, and improved screw pile founding conditions should not be assumed to be present at greater depth.

7.6.4 Negative Skin Friction

It is suggested that an allowance be made for the effects of negative skin friction on the shafts of the piles due to the placement of additional thickness of filling over the site. This is due to consolidation and creep of the compressible Holocene muds encountered below about 1 m depth.

Such negative skin friction loads should be applied to the whole of the pile shaft length to the base of the Holocene mud (i.e. to about 6 m depth in the CPTs).

The un-factored ultimate negative skin friction (τ_u) along the pile shaft may be calculated as follows:

i) in additional or existing filling, or stiff clay:

$\tau_u = 0.3\ p'$ (kPa per unit area of pile shaft)
where p' is the effective applied overburden pressure

ii) in the underlying firm or soft Holocene mud:

$\tau_u = 0.15\ p'$ (kPa per unit area of pile shaft) where p' is the effective applied overburden pressure

It is recommended that a bulk unit weight of 20 kN/m^3 be adopted for soils or filling above the groundwater level and a submerged unit weight of 10 kN/m^3 for soils below the groundwater level. For negative skin friction calculation purposes only, it is suggested that a groundwater level be conservatively assumed at approximately 1.5 m depth.

7.7 Acid Sulfate Soils

Testing of 27 ASS samples obtained from the test pits was undertaken as described in Section 6.2.

The following comments are made with reference to the QASSIT Guidelines (Ahern, C.R., Ahern, M.R. and Powell, B., 1998) and The Soil Management Guidelines (Dear, et al., 2014), and with reference to the summary of test results presented in Table 2.

- pH_F values were between 7.2 and 8.4. Where $pH_F > 4$, this indicates that if ASS are present they have not oxidised and AASS conditions are not present. Because none of the pH_F results were less than 4, AASS conditions are not likely.
- pH_{FOX} values from the screening tests are between 0.7 and 9.2. Where $pH_{FOX} < 3$, there is a strong reaction to peroxide, and pH_{FOX} is at least one pH unit below pH_F, this is a strong indicator of PASS conditions. Five tests recorded pH_{FOX} results of less than 3 with sufficiently strong reactions and decrease from pH_F to indicate PASS (all five samples were collected below 1.5 m depth in the pits).

The calculated 'existing plus potential' acidity for the eight samples submitted for chromium suite tests is summarised in Table 2. In general, the methods for determining 'existing plus potential' acidity have been derived from the Soil Management Guidelines (Dear, et al., 2014), and can be summarised as follows:

$$\text{Existing plus Potential Acidity} = \text{Potential Acidity } (S_{CR}) + \text{Actual Acidity (TAA)}$$

It is noted that acid neutralising capacity (ANC) was indicated in each of the samples tested for the chromium suite. Based on the Soil Management Guidelines (Dear, et al., 2014), the calculated 'existing plus potential' acidity can be reduced by the subtraction of the ANC term (divided by an appropriate 'fineness factor'), under certain conditions. These conditions essentially comprise the following:

- presence of carbonate material which is fine enough to buffer effectively (ie <0.5 mm); and/or
- presence of clay minerals which have a predominant carbonate content (such as residual limestone).

The ANC has been conservatively not included in the 'existing plus potential' acidity calculation for these samples, as some proportion of the ANC was likely to be from shells of greater than 0.5 mm size. Hence, for these particular results, the ANC has not been relied upon.

For less than 1000 t of soil disturbance, as is expected to be the case for this project, the action 'existing plus potential' criterion which triggers a requirement for ASS management depends upon soil type. As indicated in the Soil Management Guidelines (Dear, et al., 2014), for the predominately cohesive material encountered during the geotechnical investigation, the action criterion an 'existing plus potential' acidity of equal to or greater than 0.1% sulfur.

Of the eight samples submitted for Chromium Suite testing, six samples returned 'existing plus potential' acidity values of less than the action criterion, and two samples returned an 'existing plus potential' acidity values in excess of the action criterion, as summarised in Table 4.

Table 4: Summary of Action Criterion Exceedances

Test Location	Depth (m)	Material	'Existing plus Potential' Acidity (% S)
Pit 1	2.0	Organic silty CLAY	1.7
Pit 3	1.75	Organic silty CLAY	1.3

As such, an acid sulfate soils management plan (ASSMP) would not be required for the proposed development with disturbance up to a maximum of about 1.5 m depth below the existing ground surface (ie no disturbance below 0.5 m AHD). If, however, excavation is proposed below this depth, then an acid sulfate soils management plan (ASSMP) should be prepared covering the management of ASS disturbance, and should include the following comment regarding:

- bunding and drainage of areas of disturbance;
- control and monitoring of any run-off water that is collected by this bunding or drainage;
- liming rates, application methods and mixing protocols; and
- verification sampling/testing and regular inspections.

As a guide for liming rates, for the 'existing plus potential' acidity values of 1.3% and 1.7% sulfur, a liming rate of approximately 60 kg/t to 80 kg/t (assuming ag lime with a neutralising value of approximately 100%) should be considered for initial planning purposes.

8. Limitations

Douglas Partners Pty Ltd (DP) has prepared this report for this project at the corner of Sheridan and Hartley Streets, Cairns in accordance with DP's proposal 216856.00.P.001.Rev0 dated 26 July 2022 and acceptance received from Clarke and Prince Pty Ltd dated 28 July 2022. The work was carried out under DP's Conditions of Engagement. This report is provided for the exclusive use of Clarke and Prince 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 DP, does so entirely at its own risk and without recourse to DP for any loss or damage. In preparing this report DP 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 DP's field testing has been completed.

DP's advice is based upon the conditions encountered during this investigation. The accuracy of the advice provided by DP 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. DP 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 DP. This is because this report has been written as advice and opinion rather than instructions for construction.

The scope of work for this investigation/report did not include the assessment of surface or sub-surface materials or groundwater for contaminants, within or adjacent to the site. Should evidence of fill of unknown origin be noted in the report, and in particular the presence of building demolition materials, it should be recognised that there may be some risk that such fill may contain contaminants and hazardous building materials.

9. References

Ahern, C.R., Ahern, M.R. and Powell, B. (1998). *Guideline for Sampling and Analysis of Lowland Acid Sulfate Soils (ASS) in Queensland 1998.* Indooroopilly, Queensland, Australia : Queensland Department of natural Resources, Resources Sciences Centre.

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McAnally, P. A., & Douglas, D. J. (1984). The Design of Frankipiles in Clay. *4th Australian- New Zealand Conference on Geomechanics* (pp. 405-407). Perth: Australian Geomechanics Society.

Douglas Partners Pty Ltd

Appendix A

About this Report
Sampling Methods
Soil Descriptions
Symbols and Abbreviations
Cone Penetration Tests

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.

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.

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.

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).

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

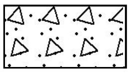
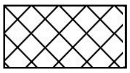
Other

fg	fragmented
bnd	band
qtz	quartz

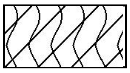
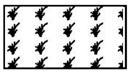
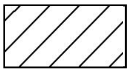
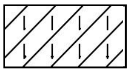
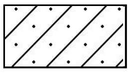
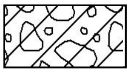
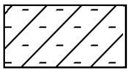
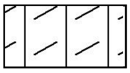
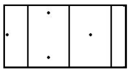
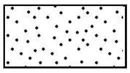
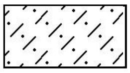
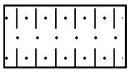
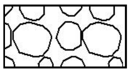
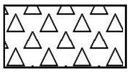
Symbols & Abbreviations

Graphic Symbols for Soil and Rock

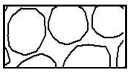
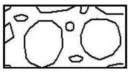
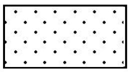
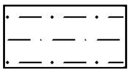
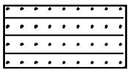
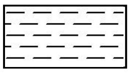
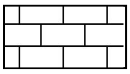
General

	Asphalt
	Road base
	Concrete
	Filling

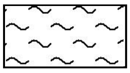
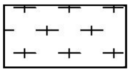
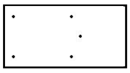
Soils

	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

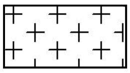
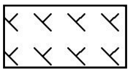
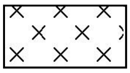
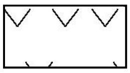
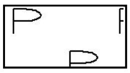
Sedimentary Rocks

	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

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

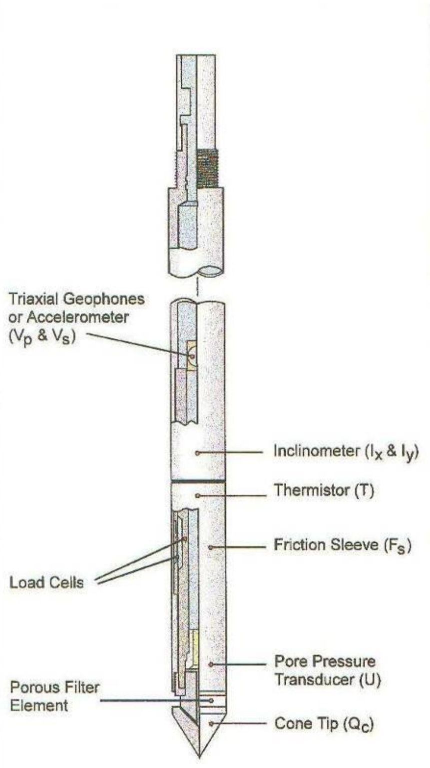


Figure 1: Cone Diagram

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)

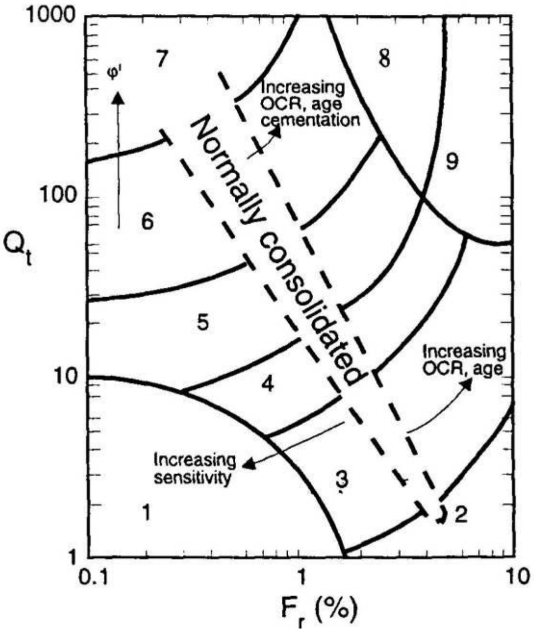


Figure 3: Soil Classification Chart

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.

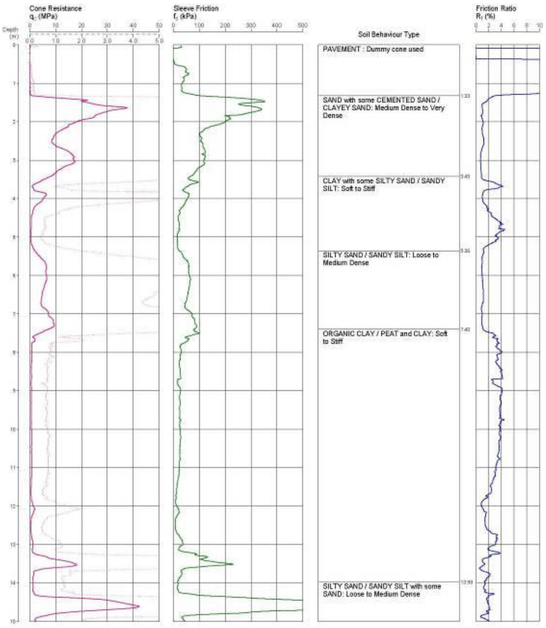
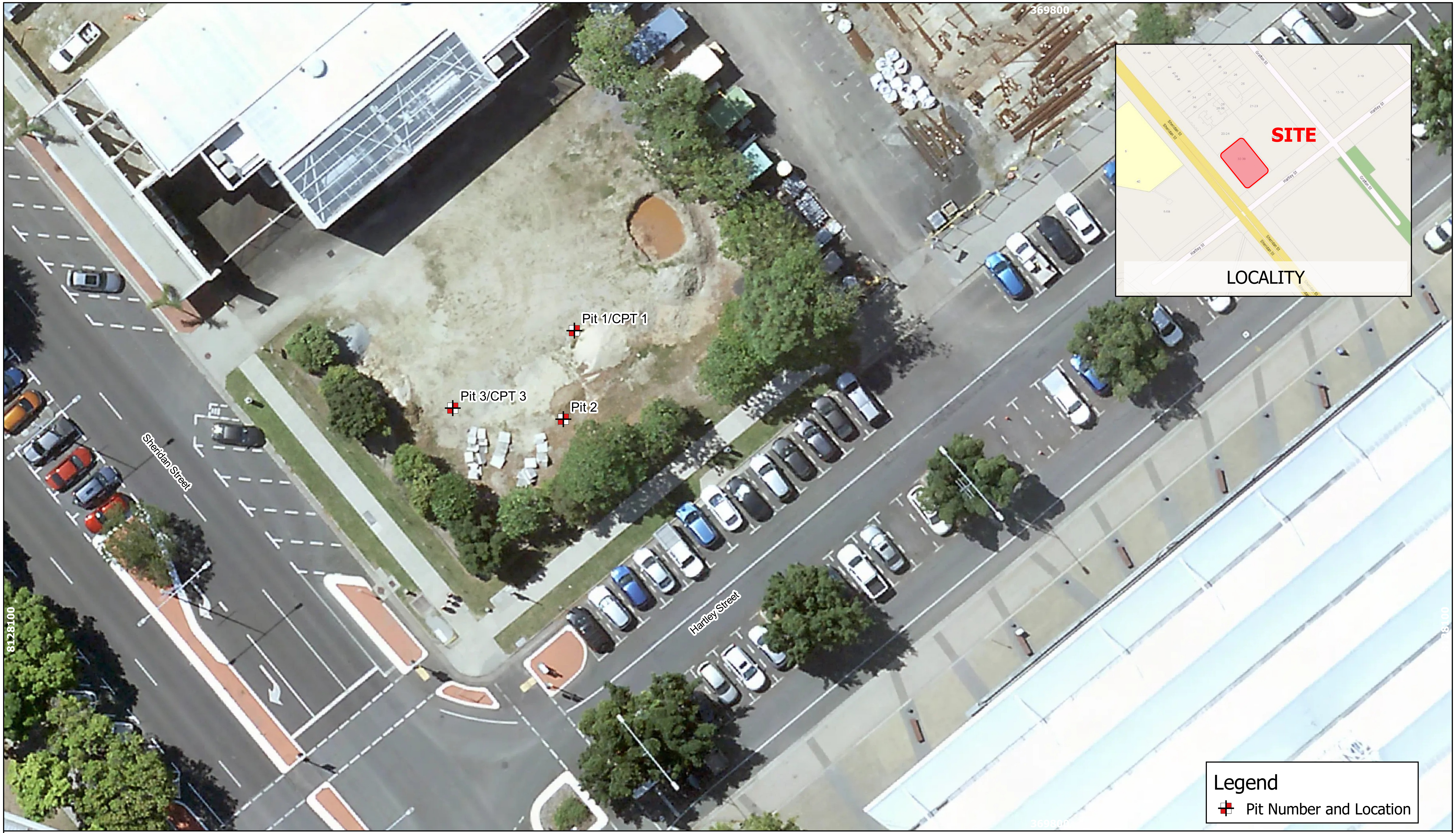


Figure 4: Sample Cone Plot

Appendix B

Drawing 1



Notes:
1. Base image from MetroMaps September 2021.
2. Locality image from StreetDirectory.com.au
3. Test locations are approximate only and are shown with reference to existing site features.

0 10 20 30 m

Douglas Partners
Geotechnics · Environment · Groundwater

CLIENT: Clarke & Prince Pty Ltd Architects
OFFICE: Cairns
DRAWN BY: CM
SCALE: As Shown
DATE: August 2022

TITLE:
Site and Test Location Plan
Proposed Ergon Depot
Cnr Sheridan and Hartley Street, Portsmith

N

PROJECT No: 216856.00
DRAWING No: 1
REVISION: 0

Appendix C

Results of Field Work

CONE PENETRATION TEST

CLIENT: Clarke & Prince Pty Ltd

PROJECT: Proposed Ergon Depot

LOCATION: cnr Sheridan and Hartley Street
Portsmith

REDUCED LEVEL: 2.0 m AHD

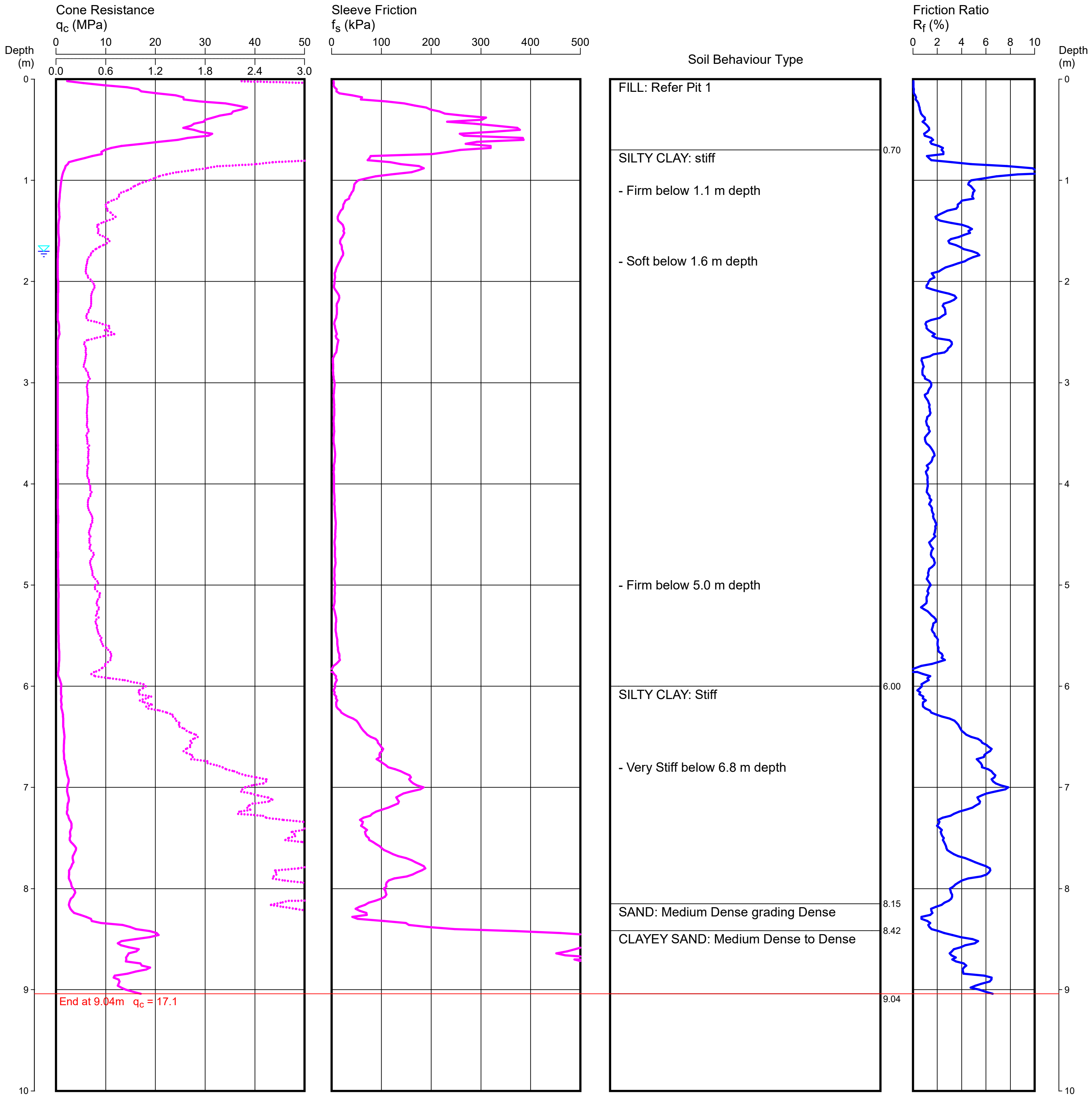
COORDINATES: 369757E 8128126N GDA 95 55K

CPT 1

Page 1 of 1

DATE 04/08/2022

PROJECT No: 216856.00



FILL: Refer Pit 1

SILTY CLAY: stiff

- Firm below 1.1 m depth

- Soft below 1.6 m depth

- Firm below 5.0 m depth

SILTY CLAY: Stiff

- Very Stiff below 6.8 m depth

SAND: Medium Dense grading Dense

CLAYEY SAND: Medium Dense to Dense

REMARKS:

Water depth after test: 1.70m depth (assumed)

File: P:\216856.00 - CAIRNS, Proposed Ergon Depot\4.0 Field Work\CPTs\CPT 1.CP5
Cone ID: 150320 **Type:** I-CFXY-10

ConePlot Version 5.9.2

Douglas Partners
Geotechnics / Environment / Groundwater

CONE PENETRATION TEST

CLIENT: Clarke & Prince Pty Ltd

PROJECT: Proposed Ergon Depot

LOCATION: cnr Sheridan and Hartley Street
Portsmith

REDUCED LEVEL: 1.9 m AHD

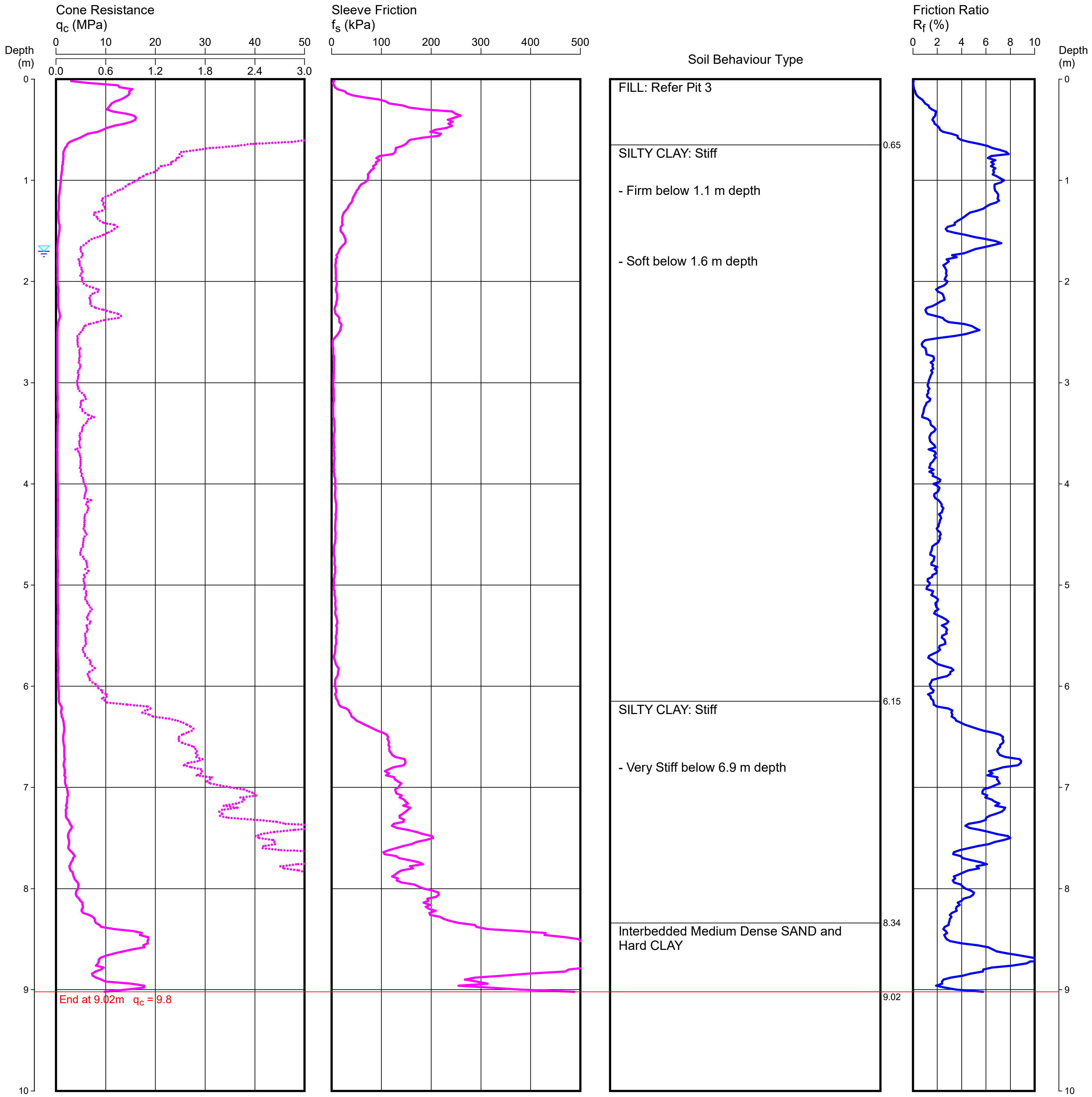
COORDINATES: 369746E 8128120N GDA 95 55K

CPT 3

Page 1 of 1

DATE 04/08/2022

PROJECT No: 216856.00



REMARKS:

Water depth after test: 1.70m depth (assumed)

File: P:\216856.00 - CAIRNS, Proposed Ergon Depot\4.0 Field Work\CPTs\CPT 3.CP5
Cone ID: 150320 **Type:** I-CFXY-10

ConePlot Version 5.9.2

Douglas Partners
Geotechnics | Environment | Groundwater

TEST PIT LOG

CLIENT: Clarke & Prince Pty Ltd Architects
PROJECT: Proposed Ergon Depot
LOCATION: Cnr Sheridan and Hartley Street, Portsmith

SURFACE LEVEL: 2.0 m AHD
EASTING: 369757
NORTHING: 8128127

PIT No: 1
PROJECT No: 216856.00
DATE: 4/8/2022
SHEET 1 OF 1

RL	Depth (m)	Description of Strata	Type	Depth	Sample	Results & Comments	Water	Dynamic Penetrometer Test (blows per 100mm) 5 10 15 20
2		Fill Silty SAND SM: fine to coarse , grey brown with fine to coarse and subangular to subrounded gravel, dry, very dense						>>
	0.27	- with shell fragments (fine sand to fine gravel size) below 0.1 m depth	D	0.25				
	0.4	- 90 mm diameter concrete fragment at 0.2 m depth	D	0.45				
	0.5	Fill Clayey Gravelly SAND SC: fine to coarse, brown, fine to coarse and subangular to subrounded gravel, moist, very dense	D	0.6		pp >600		
	0.7	Fill Silty Clayey GRAVEL GC: fine to coarse subangular to subrounded, pale yellow brown, with fine to coarse sand, moist, very dense	D	0.75				
1	1	Fill Clayey SILT ML: low plasticity, orange brown, with fine to coarse angular to subangular gravel, trace cobbles, trace fine to coarse grained sand, w<PL, hard	D	1.0		pp = 200 + ASS		1
		Silty CLAY CH: high plasticity, dark grey, w~PL, stiff, alluvial	D	1.25		pp = 100-150		
		- grading firm and trace shell fragments (fine sand to fine gravel size) below 1.1 m depth	D	1.5		pp = 100		
		- soft and dark grey mottled brown below 1.7 m depth	D	1.75			▷	
	1.9	Organic Silty CLAY OH: medium to high plasticity, dark grey, w~PL, soft, alluvial						
0	2 2.0		D	2.0				2
		Pit discontinued at 2.0m - limit of investigation						

RIG: SV 10, 10 tonne Excavator 450mm bucket

LOGGED: PJW

SURVEY DATUM: GDA94 Zone 55K

WATER OBSERVATIONS: Several localised seepage points below 1.7 m depth

REMARKS: w= Moisture content, PL= Plastic limit. Environmental (E) samples collected at depths of 0.1 m, 0.35 m, 0.45 m, 0.6 m and 1.0 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

TEST PIT LOG

CLIENT: Clarke & Prince Pty Ltd Architects
PROJECT: Proposed Ergon Depot
LOCATION: Cnr Sheridan and Hartley Street, Portsmith

SURFACE LEVEL: 2.0 m AHD
EASTING: 369756
NORTHING: 8128119

PIT No: 2
PROJECT No: 216856.00
DATE: 4/8/2022
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 100mm) 5 10 15 20
2									
	0.15	Fill Silty SAND SM: fine to coarse, grey brown with fine to coarse and subangular to subrounded gravel, with shell fragments (fine sand to fine gravel size), dry, very dense		D	0.25				
	0.4	Fill Clayey Gravelly SAND SI: fine to coarse, brown, fine to coarse and subangular to subrounded gravel, moist, very dense		D	0.45				
	0.5	Fill Clayey Gravelly SAND SC: fine to coarse, brown, fine to coarse subangular to subrounded gravel, moist, very dense			0.5		pp >600		
				D	0.6				
	0.7	Fill Gravelly Clayey SILT ML: low plasticity, orange brown, fine to coarse and angular to subangular gravel, trace fine to coarse sand, trace cobbles and boulders, w<PL, hard		D	0.75		pp = 350		
1	1	Silty CLAY CH: high plasticity, dark grey, trace shell fragment (fine sand to fine gravel size), stiff, alluvial - shells up to 20 mm diameter at 1.0 m depth		D	1.0		pp = 200		1
				D	1.25		pp = 150		
		- firm and dark grey mottled brown below 1.5 m depth		D	1.5				
	1.7	Organic Silty CLAY OH: medium to high plasticity, dark grey, w~PL, soft to firm, alluvial		D	1.75		pp = 100	▷	
0	2			D	2.0				2
	2.1	Sandy Silty CLAY CI-CH: medium to high plasticity, dark grey, fine to coarse, with shell fragments, w~PL, soft, alluvial							
	2.4	Pit discontinued at 2.4m - limit of investigation		D	2.4				

RIG: SV 10, 10 tonne Excavator 450mm bucket
LOGGED: PJW
SURVEY DATUM: GDA94 Zone 55K

WATER OBSERVATIONS: Several localised seepage points below 1.7 m depth

REMARKS: w= Moisture content, PL= Plastic limit. Environmental (E) samples collected at depths of 0.1 m, 0.30 m, 0.6 m and 1.0 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

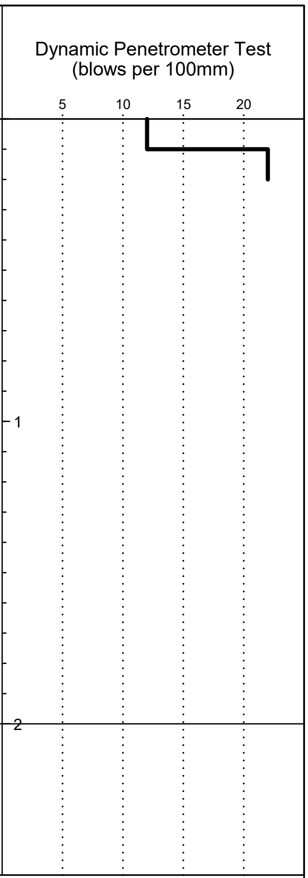
TEST PIT LOG

CLIENT: Clarke & Prince Pty Ltd Architects
PROJECT: Proposed Ergon Depot
LOCATION: Cnr Sheridan and Hartley Street, Portsmith

SURFACE LEVEL: 1.9 m AHD
EASTING: 369746
NORTHING: 8128120

PIT No: 3
PROJECT No: 216856.00
DATE: 4/8/2022
SHEET 1 OF 1

RL	Depth (m)	Description of Strata	Graphic Log	Sampling & In Situ Testing: Type	Depth	Sample	Results & Comments	Water
	0.15	Fill SAND SM: fine to coarse, yellow brown, with silt, trace fine subangular to subrounded gravel, trace cobbles, dry, very dense						
	0.25	Fill Sandy GRAVEL GP: fine angular, grey, fine to coarse sand, dry, very dense			0.3		pp >600	
	0.35	Fill Sandy Silty CLAY CI: medium plasticity, dark grey, fine to medium sand, w<PL, hard		D	0.5			
	0.65	Fill Gravelly Clayey SILT ML: low plasticity, orange brown, fine to coarse and subangular to subrounded, with cobbles and boulders, trace sand, w<PL, hard		D	0.7 0.75		pp = 350	
1	1	Silty CLAY CH: high plasticity, dark grey brown, trace shell fragments (fine sand to fine gravel size), w<PL, stiff, alluvial - grading firm from below 1.1 m depth		D	1.0		pp = 200	
				D	1.25			
				D	1.5			
		- soft below 1.6 m depth						▷
	1.7	Organic Silty CLAY OH: medium to high plasticity, dark grey, w~PL, soft, alluvial		D	1.75		pp = 50	
0	2 2.0	Pit discontinued at 2.0m - limit of investigation		D	2.0			



RIG: SV 10, 10 tonne Excavator 450mm bucket **LOGGED:** PJW **SURVEY DATUM:** GDA94 Zone 55K

WATER OBSERVATIONS: Several localised seepage points below 1.6 m depth

REMARKS: w= Moisture content, PL= Plastic limit. Environmental (E) samples collected at depths of 0.1 m, 0.2 m, 0.3 m, 0.45 m and 0.9 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 Results

Material Test Report

Douglas Partners
Geotechnics / Environment / Groundwater
Douglas Partners Pty Ltd
Townsville Laboratory
29 Civil Road Garbutt QLD 4814
Phone: (07) 4779 9866
Email: Townsville@douglaspartners.com.au

Accredited for compliance with ISO/IEC 17025 - Testing

Approved Signatory: Rachel Moss
Lab Manager
Laboratory Accreditation Number: 828

Report Number:	**216856.00-1**
Issue Number:	1
Date Issued:	26/08/2022
Client:	Clarke & Prince Pty Ltd Architects 3 Scott Street, Cairns QLD 4870
Contact:	Martin Majer
Project Number:	216856.00
Project Name:	Proposed Ergon Depot
Project Location:	cnr Sheridan and Hartley Street, Portsmith QLD
Work Request:	3022
Sample Number:	TW-3022A
Date Sampled:	08/08/2022
Dates Tested:	23/08/2022 - 25/08/2022
Sampling Method:	Sampled by Engineering Department *The results apply to the sample as received*
Sample Location:	**TP1 , Depth: 0.6**

Atterberg Limit (AS1289 3.9.2 & 3.2.1 & 3.3.2)		Min	Max
Sample History	Oven Dried		
Preparation Method	Dry Sieve		
Retained 0.425 (%)			
Liquid Limit (%)	29		
Plastic Limit (%)	24		
Plasticity Index (%)	**5**		

Linear Shrinkage (AS1289 3.4.1)		Min	Max
Moisture Condition Determined By	AS 1289.3.1.2		
Linear Shrinkage (%)	1.0		
Cracking Crumbling Curling	None		

Moisture Content (AS 1289 2.1.1)	
Moisture Content (%)	9.9

Material Test Report

Douglas Partners
Geotechnics / Environment / Groundwater
Douglas Partners Pty Ltd
Townsville Laboratory
29 Civil Road Garbutt QLD 4814
Phone: (07) 4779 9866
Email: Townsville@douglaspartners.com.au

Accredited for compliance with ISO/IEC 17025 - Testing

Approved Signatory: Rachel Moss
Lab Manager
Laboratory Accreditation Number: 828

Report Number:	**216856.00-1**
Issue Number:	1
Date Issued:	26/08/2022
Client:	Clarke & Prince Pty Ltd Architects 3 Scott Street, Cairns QLD 4870
Contact:	Martin Majer
Project Number:	216856.00
Project Name:	Proposed Ergon Depot
Project Location:	cnr Sheridan and Hartley Street, Portsmith QLD
Work Request:	3022
Sample Number:	TW-3022B
Date Sampled:	08/08/2022
Dates Tested:	23/08/2022 - 25/08/2022
Sampling Method:	Sampled by Engineering Department *The results apply to the sample as received*
Sample Location:	**TP2 , Depth: 1.0**

Atterberg Limit (AS1289 3.1.2 & 3.2.1 & 3.3.1)		Min	Max
Sample History	Oven Dried		
Preparation Method	Dry Sieve		
Liquid Limit (%)	60		
Plastic Limit (%)	21		
Plasticity Index (%)	**39**		

Linear Shrinkage (AS1289 3.4.1)		Min	Max
Moisture Condition Determined By	AS 1289.3.1.2		
Linear Shrinkage (%)	15.5		
Cracking Crumbling Curling	Cracking & Curling		

Moisture Content (AS 1289 2.1.1)	
Moisture Content (%)	33.7

ANALYTICAL REPORT

Accreditation No. 2562

CLIENT DETAILS

Contact	Dan Martin
Client	DOUGLAS PARTNERS PTY LTD
Address	NATIONAL ACCOUNTS PAYABLE PO BOX 472 WEST RYDE NSW 2114
Telephone	07 4055 1550
Facsimile	07 4055 1774
Email	dan.martin@douglaspartners.com.au
Project	**216856 - Proposed Ergon Depot**
Order Number	(Not specified)
Samples	8

LABORATORY DETAILS

Manager	Anthony Nilsson
Laboratory	SGS Cairns Environmental
Address	Unit 2, 58 Comport St Portsmith QLD 4870
Telephone	+61 07 4035 5111
Facsimile	+61 07 4035 5122
Email	AU.Environmental.Cairns@sgs.com
SGS Reference	**CE160960 R0**
Date Received	09 Aug 2022
Date Reported	22 Aug 2022

COMMENTS

Accredited for compliance with ISO/IEC 17025 - Testing. NATA accredited laboratory 2562(3146/19038)

SIGNATORIES

Anthony NILSSON
Operations Manager

		Sample Number	CE160960.001	CE160960.002	CE160960.003	CE160960.004
		Sample Matrix	Soil	Soil	Soil	Soil
		Sample Name	TP1 0.6m	TP2 0.45m	TP3 0.3m	TP1 1.25m
Parameter	Units	LOR				

Moisture Content Method: AN002 Tested: 9/8/2022

% Moisture	%w/w	0.5	**8.1**	**6.6**	**11**	**27**

TAA (Titratable Actual Acidity) Method: AN219 Tested: 11/8/2022

pH KCl	pH Units	-	**8.2**	**9.1**	**8.7**	**8.5**
Titratable Actual Acidity	kg H2SO4/T	0.25	<0.25	<0.25	<0.25	<0.25
Titratable Actual Acidity (TAA) moles H+/tonne	moles H+/T	5	<5	<5	<5	<5
Titratable Actual Acidity (TAA) S%w/w	%w/w S	0.01	<0.01	<0.01	<0.01	<0.01

Chromium Reducible Sulfur (CRS) Method: AN217 Tested: 11/8/2022

Chromium Reducible Sulfur (Scr)	%	0.005	<0.005	<0.005	<0.005	**0.035**
Chromium Reducible Sulfur (Scr)	moles H+/T	5	<5	<5	<5	**22**

Acid Neutralising Capacity (ANC) Method: AN214 Tested: 19/8/2022

Acid Neutralisation Capacity (ANCBT) as % $CaCO_3$	% CaCO3	0.1	**0.1**	**2.1**	**4.3**	**9.1**
Acid Neutralisation Capacity (ANCBT) as kg H_2SO_4/t	kg H2SO4/T	0.1	**1.2**	**21**	**42**	**89**
ANC as % $CaCO_3$	% CaCO3	0.1	**0.1**	**2.1**	**4.3**	**9.1**
Lime Equivalence	% CaCO3	0.1	**0.1**	**2.1**	**4.3**	**9.1**
Acid Neutralisation Capacity (ANCBT) as acidity units	moles H+/T	3	**25**	**430**	**860**	**1800**
Acid Neutralisation Capacity (ANCBT) as % S	%w/w S	0.005	**0.040**	**0.69**	**1.4**	**2.9**

Chromium Suite Net Acidity Calculations Method: AN220 Tested: 22/8/2022

s-Net Acidity	%w/w S	0.005	<0.005	<0.005	<0.005	<0.005
s-Net Acidity without ANC	%w/w S	0.005	<0.005	<0.005	<0.005	**0.035**
a-Net Acidity	moles H+/T	5	<5	<5	<5	<5
Liming Rate	kg CaCO3/T	0.1	<0.1	<0.1	<0.1	<0.1
Verification s-Net Acidity	%w/w S	-20	**-0.03**	**-0.46**	**-0.92**	**-1.9**
a-Net Acidity without ANCBT	moles H+/T	5	<5	<5	<5	**22**
Liming Rate without ANCBT	kg CaCO3/T	0.1	<0.1	<0.1	<0.1	**1.7**

		Sample Number	CE160960.005	CE160960.006	CE160960.007	CE160960.008
		Sample Matrix	Soil	Soil	Soil	Soil
		Sample Name	TP2 1.5m	TP3 1.0m	TP3 1.75m	TP1 2.0m
Parameter	Units	LOR				
Moisture Content Method: AN002 Tested: 9/8/2022						
% Moisture	%w/w	0.5	**31**	**25**	**52**	**55**
TAA (Titratable Actual Acidity) Method: AN219 Tested: 11/8/2022						
pH KCl	pH Units	-	**7.4**	**8.4**	**6.8**	**6.8**
Titratable Actual Acidity	kg H2SO4/T	0.25	<0.25	<0.25	<0.25	<0.25
Titratable Actual Acidity (TAA) moles H+/tonne	moles H+/T	5	<5	<5	<5	<5
Titratable Actual Acidity (TAA) S%w/w	%w/w S	0.01	<0.01	<0.01	<0.01	<0.01
Chromium Reducible Sulfur (CRS) Method: AN217 Tested: 11/8/2022						
Chromium Reducible Sulfur (Scr)	%	0.005	**0.007**	<0.005	**1.3**	**1.7**
Chromium Reducible Sulfur (Scr)	moles H+/T	5	<5	<5	**822**	**1055**
Acid Neutralising Capacity (ANC) Method: AN214 Tested: 19/8/2022						
Acid Neutralisation Capacity (ANCBT) as % $CaCO_3$	% CaCO3	0.1	**0.4**	**7.7**	**1.6**	**1.4**
Acid Neutralisation Capacity (ANCBT) as kg H_2SO_4/t	kg H2SO4/T	0.1	**3.7**	**75**	**16**	**14**
ANC as % $CaCO_3$	% CaCO3	0.1	**0.4**	**7.7**	**1.6**	**1.4**
Lime Equivalence	% CaCO3	0.1	**0.4**	**7.7**	**1.6**	**1.4**
Acid Neutralisation Capacity (ANCBT) as acidity units	moles H+/T	3	**76**	**1500**	**330**	**280**
Acid Neutralisation Capacity (ANCBT) as % S	%w/w S	0.005	**0.12**	**2.5**	**0.53**	**0.45**
Chromium Suite Net Acidity Calculations Method: AN220 Tested: 22/8/2022						
s-Net Acidity	%w/w S	0.005	<0.005	<0.005	**0.97**	**1.4**
s-Net Acidity without ANC	%w/w S	0.005	**0.007**	<0.005	**1.3**	**1.7**
a-Net Acidity	moles H+/T	5	<5	<5	**600**	**870**
Liming Rate	kg CaCO3/T	0.1	<0.1	<0.1	**45**	**65**
Verification s-Net Acidity	%w/w S	-20	**-0.07**	**-1.6**	**0.97**	**1.4**
a-Net Acidity without ANCBT	moles H+/T	5	<5	<5	**820**	**1100**
Liming Rate without ANCBT	kg CaCO3/T	0.1	<0.1	<0.1	**62**	**79**

MB blank results are compared to the Limit of Reporting
LCS and MS spike recoveries are measured as the percentage of analyte recovered from the sample compared the the amount of analyte spiked into the sample.
DUP and MSD relative percent differences are measured against their original counterpart samples according to the formula : *the absolute difference of the two results divided by the average of the two results as a percentage.* Where the DUP RPD is 'NA' , the results are less than the LOR and thus the RPD is not applicable.

Acid Neutralising Capacity (ANC) Method: ME-(AU)-[ENV]AN214

Parameter	QC Reference	Units	LOR	MB	LCS %Recovery
Acid Neutralisation Capacity (ANCBT) as % $CaCO_3$	LB106274	% CaCO3	0.1	<0.1	101%
Acid Neutralisation Capacity (ANCBT) as kg H_2SO_4/t	LB106274	kg H2SO4/T	0.1	<0.1	NA
ANC as % $CaCO_3$	LB106274	% CaCO3	0.1	<0.1	NA
Lime Equivalence	LB106274	% CaCO3	0.1	<0.1	

Chromium Reducible Sulfur (CRS) Method: ME-(AU)-[ENV]AN217

Parameter	QC Reference	Units	LOR	MB	LCS %Recovery
Chromium Reducible Sulfur (Scr)	LB105980	%	0.005	<0.005	85%
Chromium Reducible Sulfur (Scr)	LB105980	moles H+/T	5	<5	

TAA (Titratable Actual Acidity) Method: ME-(AU)-[ENV]AN219

Parameter	QC Reference	Units	LOR	MB	DUP %RPD	LCS %Recovery
pH KCl	LB105977	pH Units	-	6.2	0%	101%
Titratable Actual Acidity	LB105977	kg H2SO4/T	0.25	<0.25	0%	NA
Titratable Actual Acidity (TAA) moles H+/tonne	LB105977	moles H+/T	5	<5	0%	105%
Titratable Actual Acidity (TAA) S%w/w	LB105977	%w/w S	0.01	<0.01	0%	106%

METHOD	METHODOLOGY SUMMARY
AN002	The test is carried out by drying (at either 40°C or 105°C) a known mass of sample in a weighed evaporating basin. After fully dry the sample is re-weighed. Samples such as sludge and sediment having high percentages of moisture will take some time in a drying oven for complete removal of water.
AN214	Acid Neutralising Capacity (ANC)or Neutralising Value (NV): The crushed or as received sample is reacted with excess normal acid (HCl) and then back titrated with standard sodium hydroxide to determine the acid consumed. The result is expressed as kg H2SO4/tonne or %CaCO3. Based on AS4969-13.
AN217	Dried pulped sample is mixed with acid and chromium metal in a rapid distillation unit to produce hydrogen sulfide (H2S) which is collected and titrated with iodine (I2(aq)) to measure SCR.
AN219	Dried pulped sample is extracted for 4 hours in a 1 M KCl solution. The ratio of sample to solution is 1:40. The extract is titrated for acidity. Calcium, magnesium, and sulfur are determined by ICP-AES.
AN220	Chromium Suite: Scheme for the calculation of net acidities and liming rates using a Fineness Factor of 1.5.

FOOTNOTES

IS	Insufficient sample for analysis.	LOR	Limit of Reporting
LNR	Sample listed, but not received.	↑↓	Raised or Lowered Limit of Reporting
*	NATA accreditation does not cover the performance of this service.	QFH	QC result is above the upper tolerance
		QFL	QC result is below the lower tolerance
**	Indicative data, theoretical holding time exceeded.	-	The sample was not analysed for this analyte
***	Indicates that both * and ** apply.	NVL	Not Validated

Unless it is reported that sampling has been performed by SGS, the samples have been analysed as received.
Solid samples expressed on a dry weight basis.

Where "Total" analyte groups are reported (for example, Total PAHs, Total OC Pesticides) the total will be calculated as the sum of the individual analytes, with those analytes that are reported as <LOR being assumed to be zero. The summed (Total) limit of reporting is calcuated by summing the individual analyte LORs and dividing by two. For example, where 16 individual analytes are being summed and each has an LOR of 0.1 mg/kg, the "Totals" LOR will be 1.6 / 2 (0.8 mg/kg). Where only 2 analytes are being summed, the " Total" LOR will be the sum of those two LORs.

Some totals may not appear to add up because the total is rounded after adding up the raw values.

If reported, measurement uncertainty follow the ± sign after the analytical result and is expressed as the expanded uncertainty calculated using a coverage factor of 2, providing a level of confidence of approximately 95%, unless stated otherwise in the comments section of this report.

Results reported for samples tested under test methods with codes starting with ARS-SOP, radionuclide or gross radioactivity concentrations are expressed in becquerel (Bq) per unit of mass or volume or per wipe as stated on the report. Becquerel is the SI unit for activity and equals one nuclear transformation per second.
Note that in terms of units of radioactivity:

a. 1 Bq is equivalent to 27 pCi
b. 37 MBq is equivalent to 1 mCi

For results reported for samples tested under test methods with codes starting with ARS-SOP, less than (<) values indicate the detection limit for each radionuclide or parameter for the measurement system used. The respective detection limits have been calculated in accordance with ISO 11929.

The QC and MU criteria are subject to internal review according to the SGS QAQC plan and may be provided on request or alternatively can be found here: www.sgs.com.au/en-gb/environment-health-and-safety.