Douglas Partners
Geotechnics / Environment / Groundwater

Integrated Practical Solutions

Report on
Geotechnical Investigation

Proposed Community Centre
24 Treelands Drive, Yamba

Prepared for
Northrop Consulting Engineers Pty Ltd

Project 209696.00
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Douglas Partners
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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
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1. Introduction

This report presents the results of a geotechnical investigation undertaken for a proposed community centre at 24 Treelands Drive, Yamba. The investigation was commissioned in an email dated 18 November 2022 by Ross Jeans of Northrop Consulting Engineers Pty Ltd and was undertaken in accordance with Douglas Partners' proposal 209696.00.P.001.Rev0 dated 7 October 2021.

It is understood that the proposed development of the site includes demolition of the existing Treelands Drive Community Centre and construction of a new single-storey building to house a public library, art gallery and dedicated youth space. It is understood that the proposed building is to comprise a steel framed structure. A maximum 50 kN working load is anticipated for the footings supporting square hollow section columns.

The aim of the investigation was to assess the subsurface soil and groundwater conditions across the site in order to provide:

- Site classification in accordance with the requirements of AS2870.
- Recommendations on site preparation and earthworks.
- Recommendations on excavations and retaining structures.
- An appropriate foundation system for the proposed development, including an assessment of allowable bearing pressures and likely settlements.
- Suitable parameters for retaining wall design; and
- Suitable parameters for the design of new pavements.

2. Site Description

The site located at 24 Treelands Drive, Yamba. Residential properties are present to the north and east of the site. A TAFE NSW campus is present to the south. Treelands Drive and commercial properties are present to the west of the site. Ground surface levels are gently undulating across the site and range from approximately 1.9 m above Australian Height Datum (AHD) in the west to 2.9 m AHD in the east. An asphalt sealed carpark is present in the western part of the site (refer Figure 1). A single storey building is present in the central part of the site (refer Figure 2). The eastern part of the site is covered by grass (refer Figure 2). Semi-mature to mature trees are present in the northern, western and central parts of the site.

Figure 1: View north-east of the carpark in the western part of the site.

Figure 2: View west of the eastern part of the site.

3. Published Data

3.1 Geology

Reference to regional geological mapping (GSNSW, 2019) indicates that the site is underlain by Estuarine tidal delta flat deposits comprising fine to medium-grained lithic-carbonate-quartz sand (marine-deposited), silt, clay, shell material, polymictic gravel.

3.2 Hydrogeology

Reference to the publicly available groundwater monitoring bores (WaterNSW, 2021) indicates that there are two groundwater monitoring bores within 150 m of the site with records of groundwater bearing strata. Groundwater was encountered in Bore GW306558, located approximately 130 m south of the site, between 5.2 m and 6.0 m depth and a standing water level at 1.5 m depth. Bore GW306715, located approximately 140 m north of the site, recorded a standing groundwater level at 2.0 m depth. No records of groundwater bearing strata for bore GW306715 were available.

3.3 Soil Landscape

Reference to the regional soil landscape mapping (NSW Planning, Industry & Environment, 2020) indicates the site is mapped as comprising Disturbed Terrain. The soils are anticipated to deep (greater than 2 m) and variable.

3.4 Previous Geotechnical Investigation

DP undertook a geotechnical investigation of the area approximately 40 m south-east of the site for the proposed Connected Learning Centre (CLC) in 2018 (Douglas Partners, 2018). In summary, the investigation revealed that the area of the proposed CLC is underlain by fill comprising sand and silty sand to depths ranging from 0.9 m to 1.8 m overlying a thin layer (less than 200 mm thick) of firm to soft clay and medium dense sand to the limit of investigation at 2.1 m depth.

4. Field Work

4.1 Field Work Methods

The fieldwork was undertaken on 21 February 2022 and comprised three cone penetration tests (CPTs 1 to 3) to 10 m depth and two hand auger bores (Bores 4 and 5) to 2 m depth. The CPTs were undertaken using a purpose-built truck–mounted CPT rig. CPT (cone penetration testing) involves pushing an instrumented cone and friction sleeve assembly, of 35 mm diameter, into the ground. The cone was advanced at a constant rate of approximately 20 mm/second and a digital data acquisition system recorded cone tip resistance, friction sleeve resistance, inclination from vertical and encoded depth at measurement intervals of 20 mm.

The hand auger bores were supplemented with Perth sand penetrometer (PSP) tests undertaken adjacent to Bores 4 and 5 to 1.35 m and 2.1 m depth respectively. The results of the PSP tests are provided on the bore logs in Appendix B.

The location and surface level of each bore was recorded using a hand-held GPS which generally has an accuracy of ±5 m depending on satellite coverage and surrounding site conditions. The location (to MGA94) and surface elevations (in m,AHD) of each CPT presented on the CPT plots and borehole are provided in Appendix B. Ground surface levels at each bore located have been interpolated from the site survey plan.

The CPTs were set out by a geotechnical engineer from DP with reference to local features on site and detectable buried services. The approximate locations of the CPTs are shown on Drawing 1 in Appendix C.

4.2 Field Work Results

Details of the subsurface conditions from the bores and interpreted CPT results are presented in Appendix B. These should be read in conjunction with the accompanying explanatory notes, in Appendix A, which define the descriptive terms and classification methods used in the report. Table 1 below provides a summary of these subsurface conditions.

Table 1: Summary of subsurface conditions

From (m)	To (m)	Description
Surface (0.0 m)	0.65 / 1.9	Fill: fine to medium grained, sand, gravelly sand and silty sand, dark grey.
0.65 / 1.9	8.5	Sand and silty sand and some sandy silt layers. Generally medium dense with some loose and dense, or very denselayers (marine-deposited soils).
8.5 / 9.25	>10	Sand, dense (marine-deposited soils).

Free groundwater was not encountered whilst the bores remained open. The depths to groundwater, inferred from the CPT results, ranged from 1.6 m to 3.1 m depth. Groundwater levels are variable and can be affected by factors such as climatic conditions, soil permeability and tidal fluctuations.

5. Laboratory California Bearing Ratio

Laboratory California Bearing Ratio (CBR) tests were performed on two samples. CBR Detailed laboratory test result sheets are attached in Appendix C and are summarised in Table 2.

Table 2: Laboratory Test Results (Standard Compaction and CBR)

Bore	Depth (m)	Description	FMC (%)	CBR (%)	Swell (%)	MC After Soaking (%)	MC Top 30 mm (%)
4	0.1 – 0.4	Dark grey silty sand fill	5.2	8	-1.0	21.2	19.7
5	0.1 – 0.4	Dark grey silty sand fill	5.4	16	0.0	20.1	19.9

Notes to Table 2:
FMC – Field Moisture Content
CBR – California Bearing Ratio (4 day soak), vibrated compacted
Swell – Strain measured on CBR specimen after 4 days' soaking
MC – Moisture Content

6. Acid Sulfate Soil Assessment

6.1 Guidelines

This assessment was undertaken in general accordance with the following guidelines:

- Acid Sulfate Soil Management Advisory Committee (1998), Acid Sulfate Soil Manual, referred to as ASSMAC (1998).
- Qld Natural Resources, Mines and Energy (2004) Acid Sulfate Soil Laboratory Methods Guidelines.

6.2 Acid Sulfate Soil Risk Map Classification

In the event that piled foundations are to be used for the proposed buildings, in accordance with ASSMAC (1998) the site would be classed as Class 4 for works beyond 2 m below natural ground surface. Given the site location, elevation and topography, it is considered that the proposed development would have negligible long-term impact on the groundwater profile. On grade developments such as pavements and buildings with shallow spread footings will not impact the groundwater profile. As works will potentially impact soils beyond 2 m bgl during the construction of the pile footings, a review of the geomorphic setting was undertaken.

6.3 Acid Sulfate Soil Risk Mapping

Reference to the on-line Acid Sulfate Soil Risk Map (NSW Planning, Industry & Environment, 2020) indicates the site is located within a disturbed terrain where soil investigations are required to assess for acid sulfate soils. However, the areas approximately 400 m to the north-west and north-east of the site are mapped as high probability of occurrence of acid sulfate soils.

6.4 Geomorphic Setting

The likelihood of ASS occurrence at a site is a function of various geomorphic parameters ASSMAC (1998). Each is an indicator that ASS may be present on site. An assessment of the site geomorphic features is presented in Table 3.

Table 3: Site geomorphic features indicative of ASS

Geomorphic Feature	Present On Site?
Holocene sediments	Yes
Soil horizons less than 5 mAHD	Yes
Marine / estuarine sediments or tidal lakes	Yes
Coastal wetland; backwater swamps; waterlogged or scalded areas; inter-dune swales or coastal sand dunes	Possible before clearing
Dominant vegetation is mangroves, reeds, rushes and other swamp or marine tolerant species	Possible before clearing
Geologies containing sulfide bearing material / coal deposits or former marine shales/sediments	Yes
Deep older (Holocene or Pleistocene) estuarine sediments > 10 m bgl (if deep excavation or drainage is proposed)	Yes

At least five of the geomorphic features listed are present on site. Therefore, the geomorphic setting of the site indicates that ASS may be present and hence intrusive investigation, with laboratory testing of soils, was undertaken.

6.5 ASS Field Screening

6.5.1 Assessment Criteria

Initial screening comprises assessment of the field pH (pH_f) and oxidised pH (pH_{fox}) against the following ASSMAC (1998) criteria:

- An initial soil pH (pH_f) < 4.0 is indicative of actual acid sulfate soils (AASS).
- An oxidised soil pH (pH_{fox}) < 3.5 is indicative of potential acid sulfate soils (PASS).
- Where $pH_f - pH_{fox} > 1$ and:
  - pH_{fox} is 3.5 – 4, the soil is likely PASS.
  - pH_{fox} is 4 – 5, it is neither a positive or negative indicator of potential PASS (neutral PASS).
  - pH_{fox} > 5, with little to no drop in pH, it is indicative of little net acid generation ability and is unlikely PASS.

6.5.2 Methodology

Eight representative samples were selected from Bores 4 and 5 and submitted for field ASS screening (pH_f and pH_{fox}). Table 4 provides a summary of the field screening test results. Laboratory test reports are provided in Appendix D.

Table 4: Summary of the field ASS screening results and assessment

Bore	Sample Depth (m)	Material	pH_f	pH_{fox}	ΔpH	Potential Classification
4	0.5	Fill	8.3	6.3	2.0	Unlikely PASS
	1.0	Fill	9.2	6.6	2.6	Unlikely PASS
	1.5	Fill	9.2	6.5	2.7	Unlikely PASS
	2.0	Marine sand	8.9	6.4	2.5	Unlikely PASS
5	0.5	Fill	9.2	6.5	2.7	Unlikely PASS
	1.0	Fill	8.2	5.8	2.4	Unlikely PASS
	1.5	Fill	8.7	6.3	2.5	Unlikely PASS
	2.0	Marine sand	8.3	6.0	2.3	Unlikely PASS

6.6 Conclusion

In summary, the test results indicate that the sampled fill and marine sand are unlikely to be PASS. It is possible ASS may be present beyond 2 m depth below groundwater level. An ASS management plan is not considered necessary.

If piles are to be adopted in the design and construction of the proposed development, it is recommended further assessment be undertaken to determine the potential for ASS below 2 m depth and the requirements for an ASS management plan.

7. Geotechnical Comments

7.1 Site Classification

In the absence of verification testing, the fill is presumed to be 'uncontrolled'. Based on the ground condition encountered, the site is classed as Class 'P' in accordance with AS2870 2011 due to the presence of uncontrolled fill extending to 1.9 m depth.

Class S may be adopted provided the uncontrolled fill is removed and replaced with Level 1 controlled fill of low reactivity (i.e. clean sand).

7.2 Footings

Footing design should consider the presence of the uncontrolled fill. The following options for the support of the proposed building have been considered, in decreasing order of future confidence in the performance of the footings:

- Piles founded in medium dense or dense natural soils below all fill;
- Shallow footings within engineered fill; or

- Shallow footings within the existing fill, following additional investigation to confirm that the fill is suitable to support the structure and any additional site preparation depending on the results of the additional investigation.

These options are discussed in the following sections.

7.2.1 Piles

The column/pier loads for the proposed building are not known at this stage. The subsurface profile encountered in the bores included uncontrolled fill to depths of up to 1.9 m underlain by loose to dense sand to the limit of investigation at 10 m depth.

Cased bored, continuous flight auger (CFA) or screw piles could be used. Owing to potential for collapsing ground conditions, groundwater inflow, bored piles would need casing to support the hole during construction. It should be noted that construction of CFA piles require careful control to avoid deviation of piles vertically, pile necking or honey-combing and requires strict quality control procedures.

A geotechnical strength reduction factor (ϕ_g) of 0.48 is suggested for low redundancy systems and a ϕ_g of 0.56 is suggested for high redundancy systems. The redundancy category should be confirmed by the designer once the level of inspection during testing, instrumentation during installation and redundancy in design is confirmed. It should be noted, however, that reference to AS 2159 (2009) indicates that where a basic geotechincal strength reduction factor of greater than 0.4 is used, testing shall be performed to verify pile serviceability and also integrity of the pile shafts. If such testing is not proposed a geotechincal strength reduction factor of 0.4 should be adopted.

The ultimate limiting end bearing and serviceability criteria given in Table 5 may be used to assess the limiting states for pile design purposes in accordance with AS 2159:2009.

The settlement of piles subjected to vertical loads will vary depending on the ('serviceability' or working) loads applied and the subsurface conditions below the pile toe.

Table 5: Preliminary Bored/CFA Pile Design Parameters and Founding Depths

Diameter (m)	Depth (m)	Founding Material	Ultimate geotechnical strength ($R_{d.ug}$) (1) (kN)	Design Geotechnical Strength Rd,g (kN) (2)
0.45	3	Medium dense sand and silty sand	200	80
	4.5	Medium dense sand and silty sand	500	200
	8.0	Dense sand (3)	700	280
0.6	3	Medium dense sand and silty sand	450	180
	4.5	Medium dense sand and silty sand	700	280
	8.0	Dense sand (3)	1,300	520

Notes to table:

(1) Bearing pressure values assume a minimum embedment of one pile diameter into the relevant bearing stratum. The bearing values should be downgraded by 50% in the case of negligible pile embedment of founding material

(2) Design geotechnical strength multiplied by geotechnical reduction factor $ø_g$ = 0.4. Serviceability end bearing parameters could experience settlements of about 1% of the pile diameter

(3) For piles founded below 8.0 m below existing ground surface level. Should piles founded within the dense sand be considered further, it is recommended that additional investigation is undertaken to confirm the depth to the strata across the site and confirm that it extends to sufficient thickness (i.e. is not underlain by weaker material).

The design geotechnical strength of a pile ($R_{d,g}$) is the ultimate geotechnical strength ($R_{d,ug}$) multiplied by the geotechnical strength reduction factor (ϕ_g), such that:

$$R_{d,g} = \phi_g . R_{d,ug}$$

The calculated design geotechnical strength ($R_{d,g}$) must equal or exceed the structural design action effect (E_d). Further reference can be made to AS 2159 (2009) regarding these terms and the design procedure.

AS 2159 (2009) recommends that a value of $\phi_g = 0.40$ should be used if no pile load testing is undertaken. If load testing is to be undertaken a higher ϕ_g can be utilised which will provide higher pile capacities than provided in Table 5.

For vertical loading, it is suggested that piles should be spaced at 2.5 pile diameters or greater such that the overall capacity of the pile group can be equivalent to the sum of the individual piles (i.e., group efficiency factor of unity).

For calculation of serviceability geotechnical strength, the capacity can be calculated using the serviceability end bearing values and ultimate shaft adhesion values. In the serviceability case, these values do not need to be factored. It is recommended that deflection under load is checked and compared to serviceability deflection limits.

The base of the bored pile holes should be clean, dry and free from loose material at the time of placing concrete. If there is water in the hole, the water should be removed or alternatively, the concrete should be placed using the tremie method. Specific cleaning buckets and grooving tools should be used in pile construction, together with suitable inspection or verification methods. It is noted, however, that casing may be required and no contribution of shaft adhesion over the length of casing should be accounted for.

Steel screw piles are a proprietary pile type and are relatively quick to install. They rely on the soil underlying the helix to resist vertical loads without undergoing excessive settlement. It is usual practice to ignore skin friction in determining the vertical capacity of screw piles. It should also be noted that the lateral capacity of steel screw piles would be negligible.

7.2.2 Shallow Footings within Engineered Fill

Shallow footings may be adopted provided they are founded within engineered fill. The uncontrolled fill could be stripped from the proposed building footprint area. The stripped surfaces should be inspected and test rolled in the presence of a geotechnical engineer. Any areas exhibiting significant deflections under test rolling must be appropriately treated at the direction of the geotechnical engineer. Approved well-graded granular fill should then be placed in layers not exceeding 300 mm loose thickness and compacted 80% density index in accordance with AS 3798:2007.

The allowable bearing pressure of footings founded in granular engineered fill are a function of footing geometry and depth of embedment. The design bearing pressure is normally selected on the basis of the need to limit settlement to tolerable levels. Local bearing pressures beneath slab edge beams or internal beams should not exceed 150 kPa. The beam footings should be 0.5 m to 1 m wide and founded at least 0.4 m below adjacent ground level. Settlements are expected to be less than 25 mm.

7.2.3 Shallow Footings on Existing Fill

The option which carries the highest risk of poor future performance includes the support of the structure on the existing fill. The investigation undertaken was limited to the northern and eastern parts of the site to minimise the impact of the current site users and where access was available for the CPT rig. Hence, adequate characterisation of the homogeneity and compaction levels within the existing fill has not been possible within the present investigation. There is therefore a possibility that whilst the fill encountered in the bores and CPTs appeared to be predominantly granular and in a moderate to well compacted condition, poorer quality fill may be present elsewhere on site. Therefore, prior to adoption of footings within the existing fill, significant additional investigation should be undertaken to provide greater confidence in the consistency and strength of the fill profile, and should include the following:

- Test bores with in-situ testing across the footprint of the proposed building (say 5 bores with standard penetration tests) to assess the condition of the existing fill and underlying natural soils; and
- PSP testing (say 10 test locations within the proposed building footprint) to assess the condition of the existing fill and underlying natural soils.

Provided favourable results are obtained from the further investigation, high level footings may be suitable on the existing fill and could be proportioned for an allowable bearing capacity of 75 kPa. Higher capacities may be achievable depending on the results of the further investigation.

In addition to the above investigations, at the time of construction all pad footing locations should be inspected by a geotechnical engineer and PSP tests undertaken to confirm the suitability of the exposed conditions for the design allowable bearing capacity.

7.3 Earthworks and Site Preparation

7.3.1 Trafficability

Maintaining a grassed surface at the site will assist trafficability of construction vehicles and equipment at the site. Exposed sand subgrade may be difficult for wheeled rubber tyred vehicles to access, particularly if there is a high proportion of silt in the sand.

Some measures that can be undertaken to reduce the impact of rutting / vehicle bogging in sand subgrade during the earthworks construction include:

- Retain grass cover wherever possible;
- Provide cut surfaces with a slight but even cross-gradient to assist surface drainage;
- “Seal” exposed fill surfaces and flatten (“iron out”) imperfections at the end of each work day by running over with a smooth-drum roller; and

- Provide temporary access roads with well graded rockfill as a trafficable layer.

7.3.2 Re-Use of Site Soils

The fill with organic matter to between 0.35 m and 0.4 m depth is considered unsuitable for engineering applications and should be placed in non-structural applications.

The sand and silty sand fill is considered suitable for re-use in structural fill, provided the material is appropriately compacted (refer Section 7.2.2).

7.4 Excavation Conditions

Excavation of about 0.5 m to 1 m depth may be required for footings, service trenches and pavement subgrade.

The results of the investigation indicate that the proposed excavations will typically encounter sand and silty sand fill and marine-deposited sand.

Conventional equipment such as hydraulic excavators will be adequate for excavation in these materials.

7.5 Excavation Support

The sand fill is generally medium dense and would be expected to stand unsupported in the short term, provided that it is appropriately battered. However, there would be the possibility of localised soil collapse / spalling. This may be exacerbated by prolonged exposure and adverse weather. The risk could be reduced by ensuring a short exposure period and/or flattening the batters.

The ground surface should be shaped to direct any seepage and possible surface runoff away from the slope and batter. Appropriate drains should be installed at the crest of the excavation as well as the toe of each slope to direct water away from the excavation.

Table 6 provides recommended maximum slopes for temporary and permanent cut / fill batters of up to 1 m.

Table 6: Recommended Maximum Batter Slopes in Cut/Fill

Strata	Cut / Fill Height (m)	Temporary Batters (H:V)	Permanent Batters [1] (H: V)
Uncontrolled fill / Medium dense sand	1	1.5:1	2:1
Controlled fill [(2)]	1	1.5:1	2:1

Notes to Table 6:
(1) Flatterer slopes should be adopted if access is required for maintenance purposes (3H:1V).
(2) Engineered fill placed and compacted in accordance with AS 3798:2007

All permanent unsupported slopes should be protected against erosion by vegetating the exposed surface.

Excavation support should be provided for excavations beyond 1.0 m depth unless further assessment is undertaken and advice provided.

7.6 Design CBR

The results of the laboratory testing on sand fill subgrade indicated 4-day soaked CBR values of 8% and 16%.

The results of the dynamic penetrometer tests (DPT) at the locations of Bores 4 and 5 indicated in-situ CBR ranging from 5% to 40%. These results should be treated with caution as the tests were not undertaken beneath an existing pavement on which the Austroads correlation is based.

The correlated results of all the in situ dynamic penetrometer tests have been plotted against depth and are presented in Figure 3, which also presents the soaked CBR tests for comparison purposes. The CBR values derived from DPT tests from ground level to 0.15 m depth are excluded from Figure 7 as it is assumed this material will be stripped from the site prior to construction of the access roads and sealed carparks.

Based on the results of the in-situ testing and soaked CBR test results a design CBR of 8% is recommended.

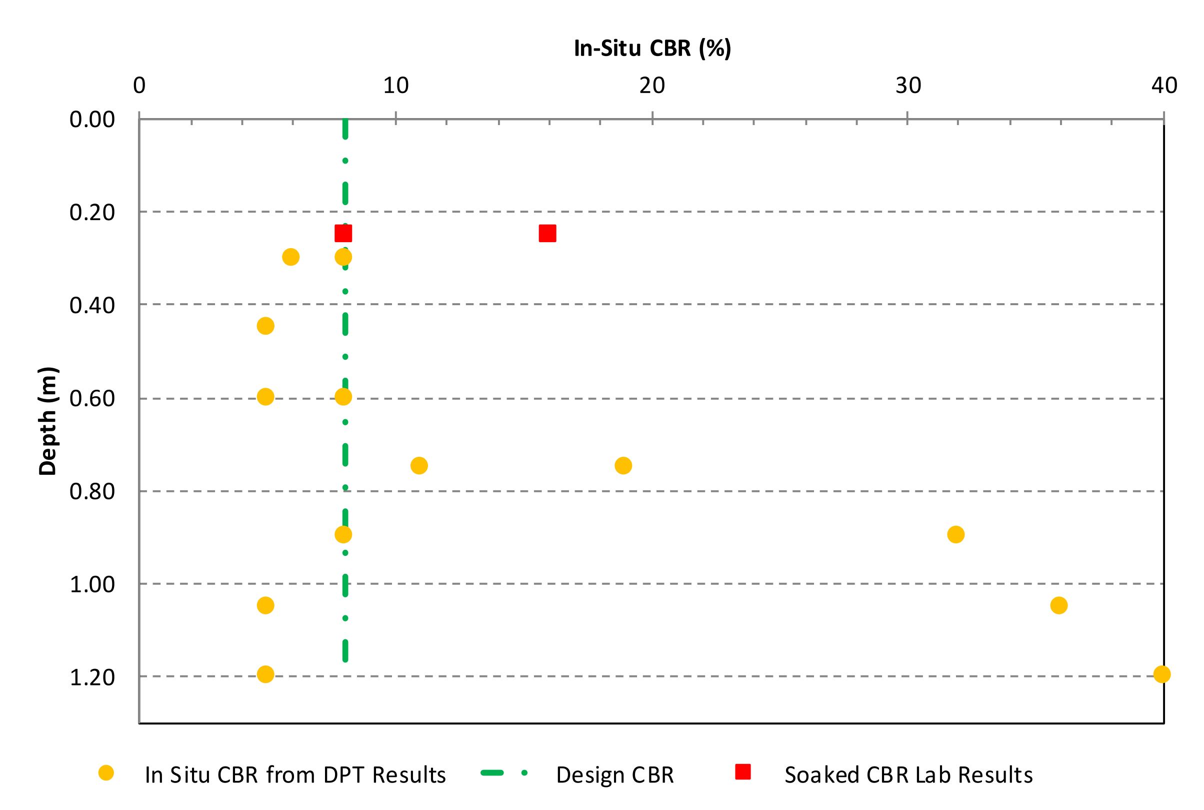

Figure 3: In situ CBR based on correlation with DPT and laboratory test results against depth.

7.7 Subgrade Preparation

Site preparation for the construction of pavement areas should include the following:

- Excavation to 0.5 m below design subgrade level and stockpile excavated fill for assessment for its suitability for re-use. It is anticipated that the fill will comprise gravelly sand and is likely to be suitable for re-use under controlled conditions;
- Remove any additional topsoil, roots, vegetation, moisture affected soils and other deleterious materials such as organic matter and / or tree affected soils from the proposed construction areas; and
- The stripped surfaces should be inspected and test rolled in the presence of a geotechnical engineer;
- Unsuitable material should be removed and replaced with approved material compacted to the project specifications;
- Any areas exhibiting significant deflections under test rolling must be appropriately treated at the direction of the geotechnical engineer.
- Approved well-graded granular fill should then be placed in layers not exceeding 300 mm loose thickness and compacted to a minimum density ratio of 95% modified with moisture contents in the range – 4 % (dry) to -1% (dry) optimum moisture content in accordance with AS 3798:2007.

7.8 Retaining Walls

For permanent retaining walls, which are free to deflect slightly, design may be based on “active” (K_a) earth pressure coefficients, assuming a triangular earth pressure distribution. This would comprise any non-propped or laterally unrestrained walls (e.g. cantilever or single propped walls).

The suggested long term (permanent) design soil parameters are shown in Table 7. Any additional surcharge loads, including those imposed by adjacent structures and inclined slopes, during or after construction, should be accounted for in design by multiplying the surcharge by the active earth pressure coefficient. Backfill placed immediately behind the wall should be free-draining (20 mm single size aggregate or coarser is suggested if sand backfill is not used) and connected to a rear wall drainage system. A slotted drainage pipe should be placed at the base of the backfill which should all be encapsulated in a geotextile fabric.

Cantilever walls should be avoided for the support any adjacent building foundations or underground services. In these areas, the wall should stiffened by designing for an at rest earth pressure coefficient (K_0), plus any surcharge from the footings if support of adjacent footings is required.

Table 7: Geotechnical Parameters for Retaining Structures (unfactored)

Parameter	Symbol	Sandy / Silty Sand fill
Bulk Density	γ	18 kN/m^3
Effective Cohesion	c’	0 kPa
Angle of Friction	ø’	30°
Active Earth Pressure Coefficient	K_a	0.33
Passive Earth Pressure Coefficient	K_p	3.0
At Rest Earth Pressure Coefficient	K_0	0.5

7.9 Site earthquake sub-soil class

The site is assessed to be a class ‘C_e (shallow soil site)’ in accordance with AS 1170.4 (2007). An earthquake Hazard Factor (z) of 0.06 may be adopted for this site.

8. References

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9. Limitations

Douglas Partners (DP) has prepared this report (or services) for this project at Treelands Drive, Yamba in accordance with DP's proposal 209696.00.P.001.Rev0 dated 7 October 2021 and acceptance received from Ross Jeans dated 18 November 2021. The work was carried out under contract No NL213021 1 December 2021. This report is provided for the exclusive use of Northrop Consulting Engineers 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.

Douglas Partners Pty Ltd

Appendix A

About This Report
Sampling Methods
Soil Descriptions
Symbols & 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.

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.

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.

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).

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

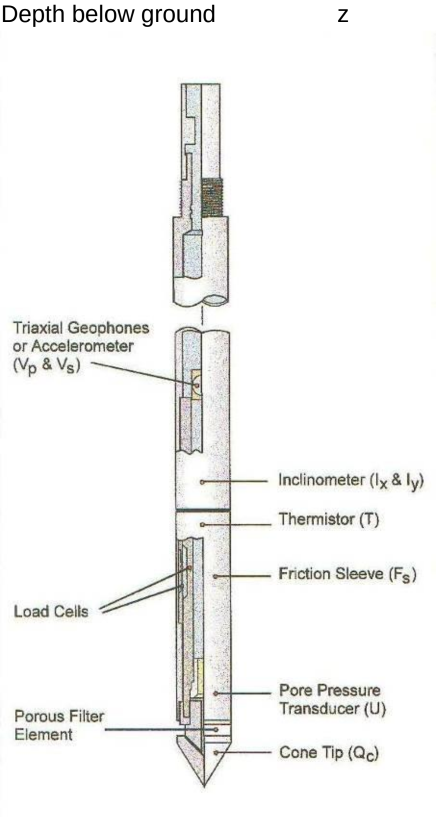


Figure 1: Cone Diagram

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)

Cone Penetration Tests

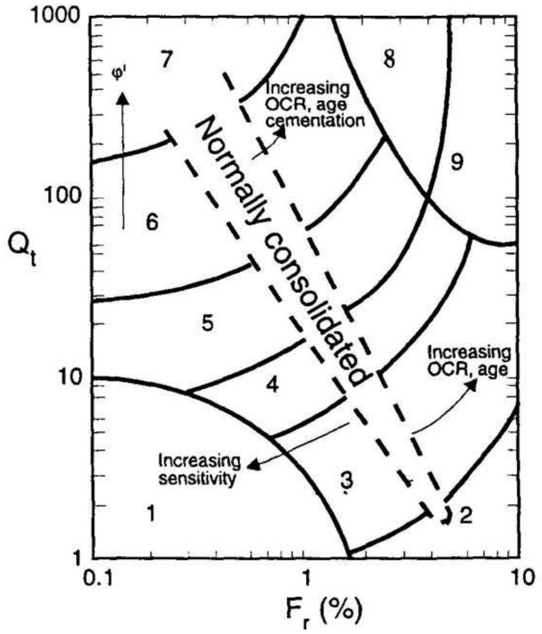


Figure 3: Soil Classification Chart

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.

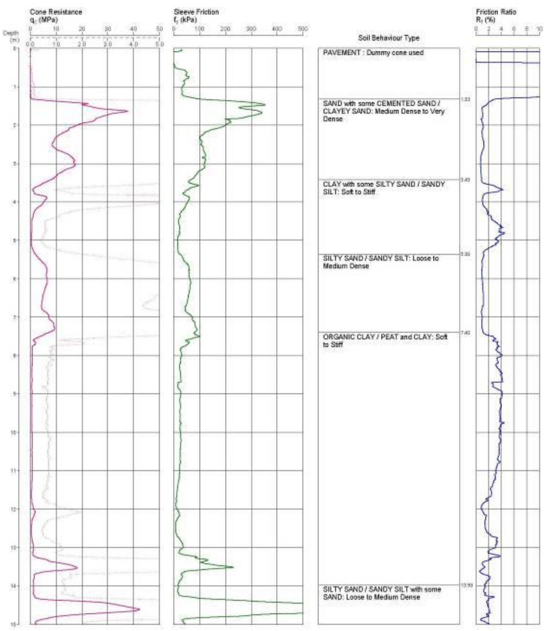


Figure 4: Sample Cone Plot

Appendix B

Cone Penetration Plots (CPT1 to CPT3)
Bore logs (Bore 4 and Bore 5)

CONE PENETRATION TEST

CLIENT: Northrop Consulting Engineers PTY LTD

PROJECT: YAMBA & MACLEAN - Community Centres

LOCATION: 24 Treelands Drive - YAMBA

REDUCED LEVEL:

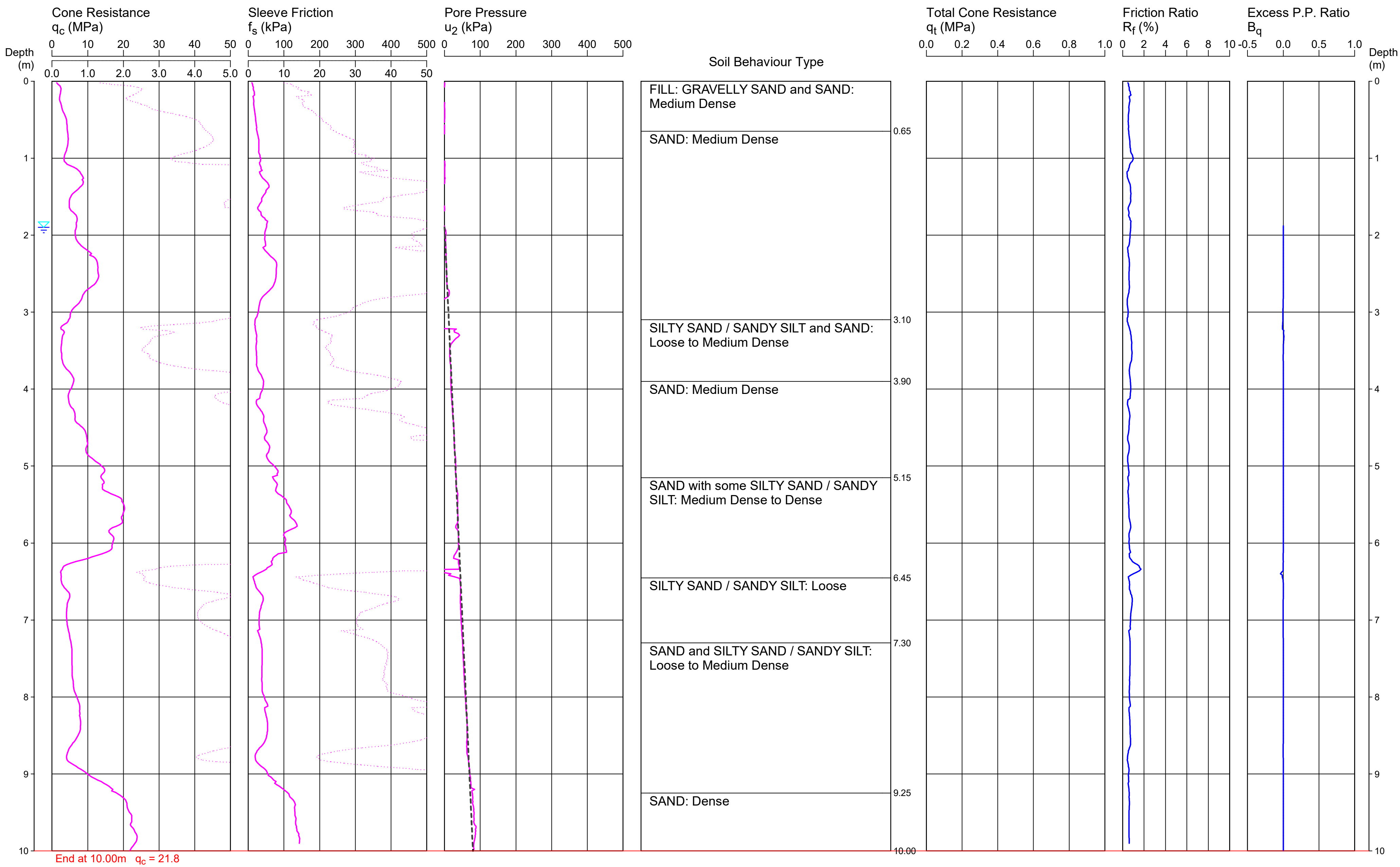
COORDINATES:

CPTu01

Page 1 of 1

DATE 21/02/2022

PROJECT No: 209696



REMARKS:

Water depth after test: 1.90m depth (assumed)

File: P:\209696.00 - YAMBA & MACLEAN, Community Centres, GEO\4.0 Field Work\CPTu\CPTu01.CP5
Cone ID: 210733 **Type:** I-CFXYP20-10

ConePlot Version 5.9.2

Douglas Partners
Geotechnics / Environment / Groundwater

CONE PENETRATION TEST

CLIENT: Northrop Consulting Engineers PTY LTD

PROJECT: YAMBA & MACLEAN - Community Centres

LOCATION: 24 Treelands Drive - YAMBA

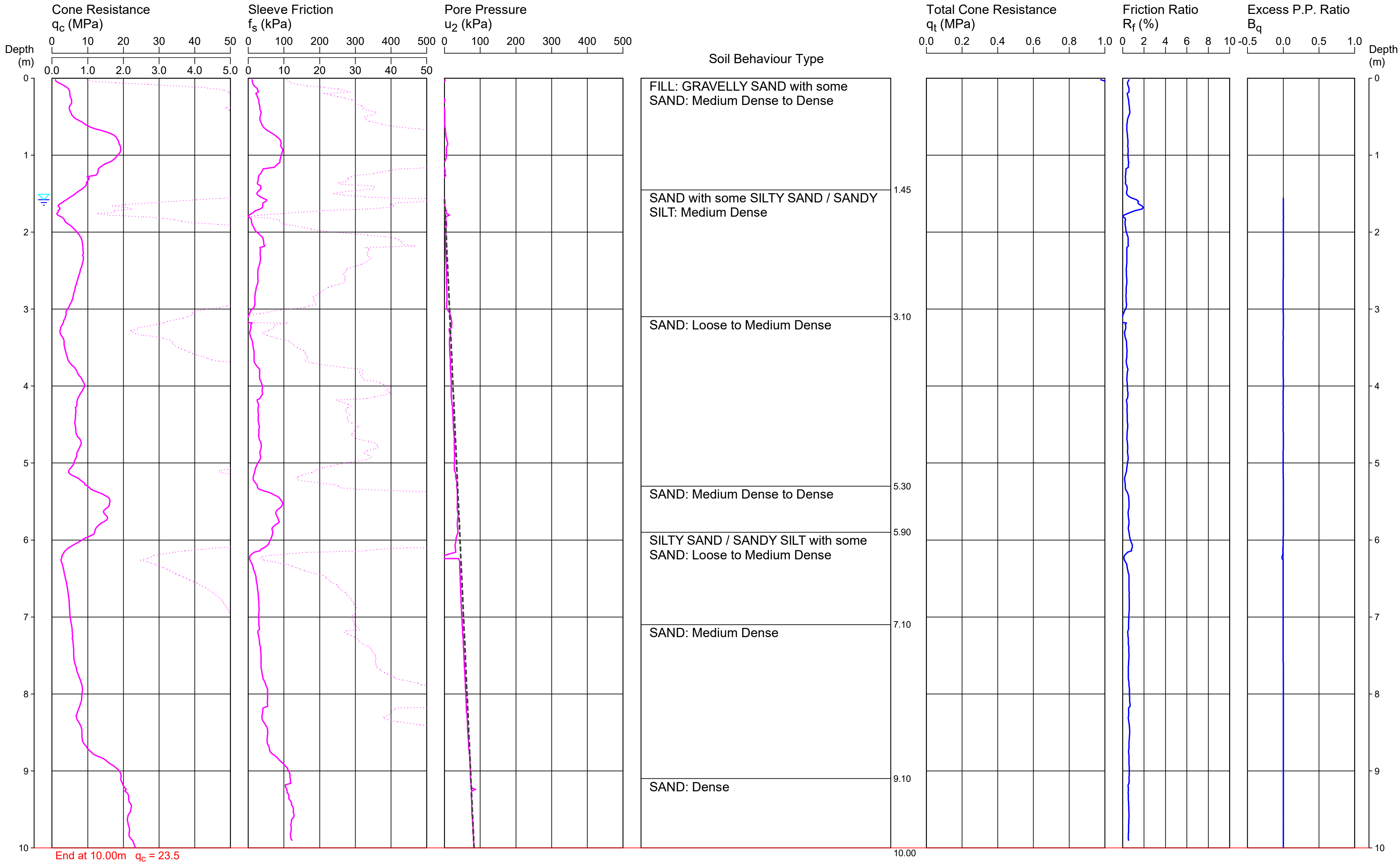
REDUCED LEVEL:

COORDINATES:

CPTu02

DATE 21/02/2022

PROJECT No: 209696



REMARKS:

Water depth after test: 1.58m depth (assumed)

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Cone ID: 120522 **Type:** I-CFXYP20-10

ConePlot Version 5.9.2

Douglas Partners
Geotechnics | Environment | Groundwater

CONE PENETRATION TEST

CLIENT: Northrop Consulting Engineers PTY LTD

PROJECT: YAMBA & MACLEAN - Community Centres

LOCATION: 24 Treelands Drive - YAMBA

REDUCED LEVEL: 2.8

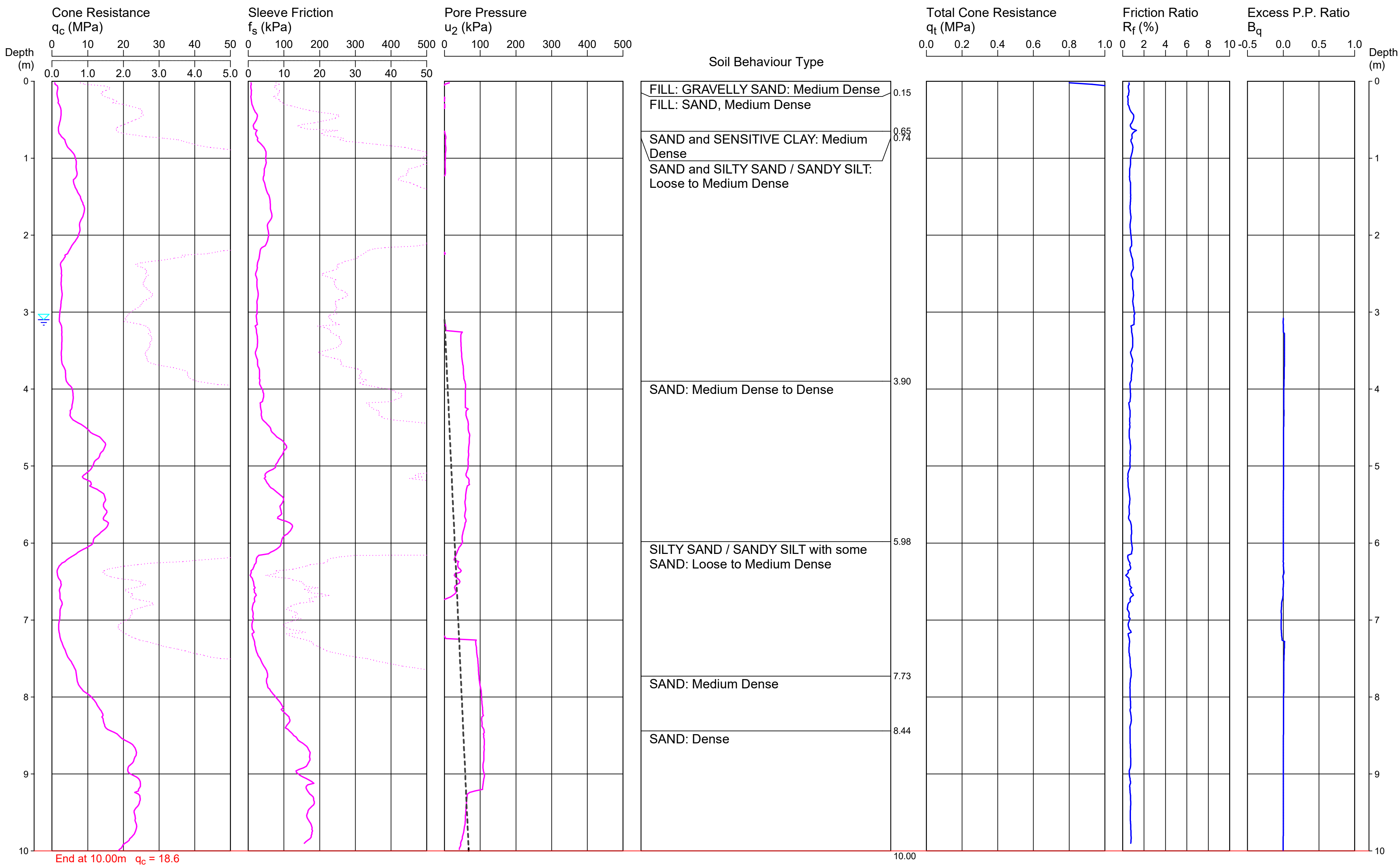
COORDINATES:

CPTu03

Page 1 of 1

DATE 21/02/2022

PROJECT No: 209696



REMARKS:

Water depth after test: 3.10m depth (assumed)

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Cone ID: 171009 **Type:** I-CFXYP20-10

ConePlot Version 5.9.2

Douglas Partners
Geotechnics | Environment | Groundwater

BOREHOLE LOG

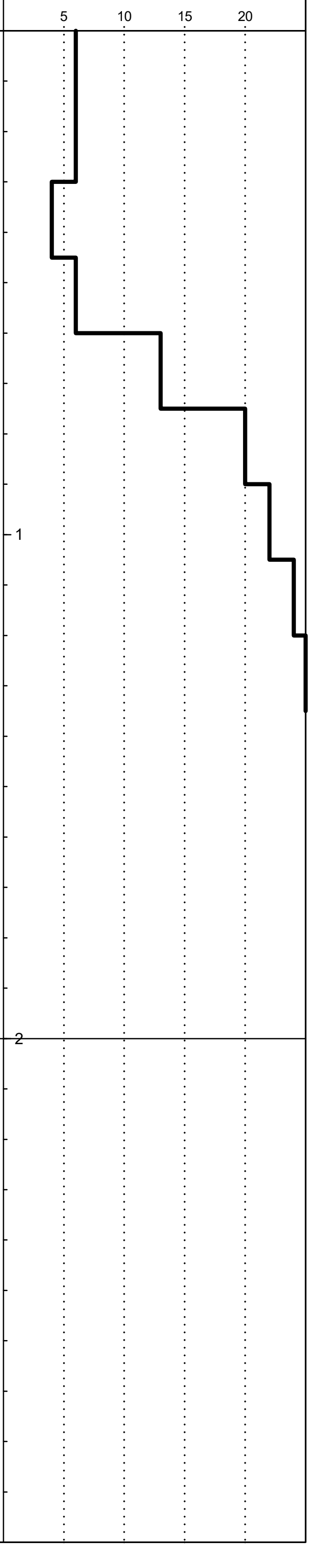
CLIENT: Northrop Consulting Engineers Pty Ltd
PROJECT: Proposed Community Centres
LOCATION: Treelands Drive, Yamba

SURFACE LEVEL: 2.9 AHD
EASTING: 531835
NORTHING: 6744779
DIP/AZIMUTH: 90°/--

BORE No: 4
PROJECT No: 209696.00
DATE: 21/2/2022
SHEET 1 OF 1

RL	Depth (m)	Description of Strata	Graphic Log	Sampling & In Situ Testing: Type	Depth	Sample	Results & Comments	Water
		FILL (Silty SAND) (SM) - fine grained, dark grey, trace rootlets, moist, with organic matter.			0.1			
				B A	0.3			
	0.4	SAND (SP) - fine to medium grained, pale grey, trace rootlets, moist, medium dense, alluvial			0.4			
				E	0.5			
		From 0.6 m, dense						
				A	0.7			
		From 0.75 m, very dense						
2								
	1			E	1.0			
				E	1.5			
1								
	2.0	From 1.95 m, pale grey mottled grey		E	2.0			
		Bore discontinued at 2.0m, limit of investigation						
0								

Dynamic Penetrometer Test (blows per 150mm)



RIG: Hand Tools **DRILLER:** Ussher **LOGGED:** Ussher **CASING:** Nil
TYPE OF BORING: 100mm ϕ Hand Auger
WATER OBSERVATIONS: No free groundwater observed
REMARKS: Handheld GPS, coordinates approximate

☒ 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: Northrop Consulting Engineers Pty Ltd
PROJECT: Proposed Community Centres
LOCATION: Treelands Drive, Yamba

SURFACE LEVEL: 2.9 AHD
EASTING: 531872
NORTHING: 6744760
DIP/AZIMUTH: 90°/--

BORE No: 5
PROJECT No: 209696.00
DATE: 21/2/2022
SHEET 1 OF 1

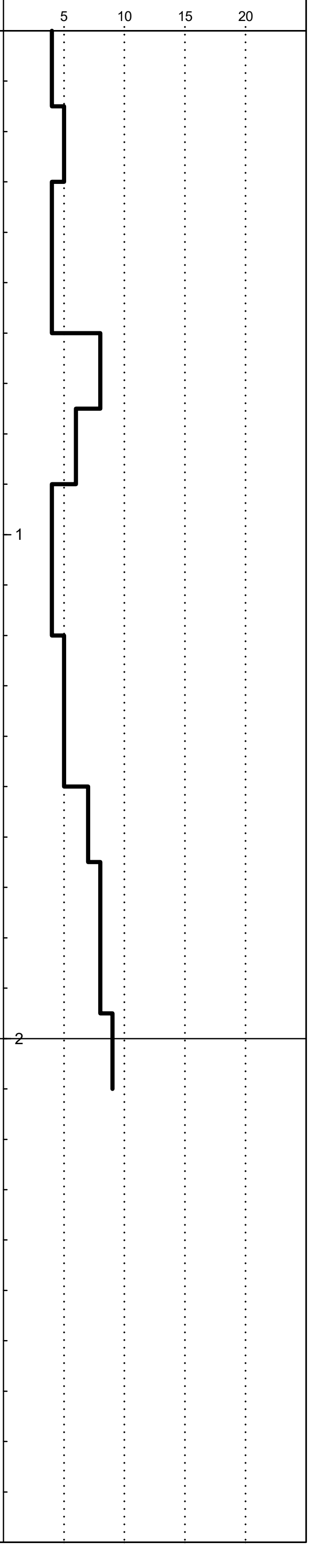
RL	Depth (m)	Description of Strata	Graphic Log
	0.0	FILL (Silty SAND) (SM) - fine to medium grained, dark grey, dry to moist, with organic matter	
	0.35	FILL (SAND) (SP) - fine to medium grained, pale grey, trace subangular to subrounded shells, moist	
2	0.9	FILL (SAND) (SP) - fine to medium grained, dark grey mottled grey, trace subangular to subrounded shells, moist From 1.3 m, grey mottled pale grey	
1	1.9	SAND (SP) - fine to medium grained, grey, moist, medium dense, alluvial	
	2.0	Bore discontinued at 2.0m, limit of investigation	
0			

Sampling & In Situ Testing

Type	Depth	Sample	Results & Comments
B	0.1–0.4		
A	0.3		
E	0.5		
A	0.7		
E	1.0		
E	1.5		
E	2.0		

Water:

Dynamic Penetrometer Test (blows per 150mm)



RIG: Hand Tools **DRILLER:** Ussher **LOGGED:** Ussher **CASING:** Nil
TYPE OF BORING: 100mm ϕ Hand Auger
WATER OBSERVATIONS: No free groundwater observed
REMARKS: Handheld GPS, coordinates approximate

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

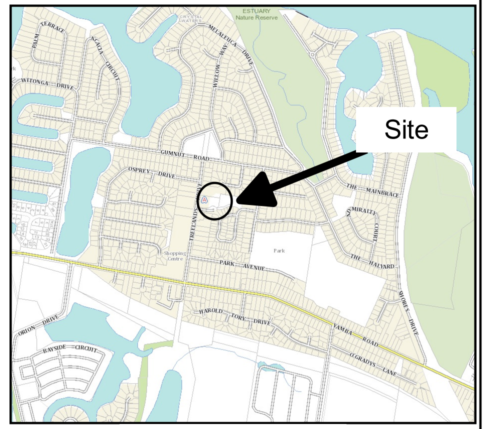
SAMPLING & IN SITU TESTING LEGEND

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 C

Drawing 1 – Test Location Plan



Site Location

Legend

- CPT
- Bore
- 2018 Investigation test locations
- Approx. Site Boundary
- Proposed building outline

0 5 10 15 20 m

Drawing adapted from Drawing 4: Ground Floor Plan and Site Plan drawn by L. Wright & P. Brennan and supplied by Northrop Consulting Engineers Pty Ltd. Image source: Metromap, image date 6 October 2021.

Test locations are approximate and have been measured using a hand held GPS with an accuracy of 5 m.

Douglas Partners Geotechnics / Environment / Groundwater	CLIENT: Northrop Consulting Engineers Pty Ltd	
	OFFICE: Port Macquarie	DRAWN BY: SvK
	SCALE: 1:500 @ A3	DATE: 16.March.2022

TITLE: **Proposed Building Footprint and Proposed Test Location Plan**
Geotechnical Investigation
Yamba Community Centre, 24 Treelands Drive, Yamba NSW

N

Project:	209696.00
DRAWING No:	1
REVISION:	0

DP.QGIS.A3LandscapeDrawingLayout.DftA

Appendix D

Laboratory Test Results

Material Test Report

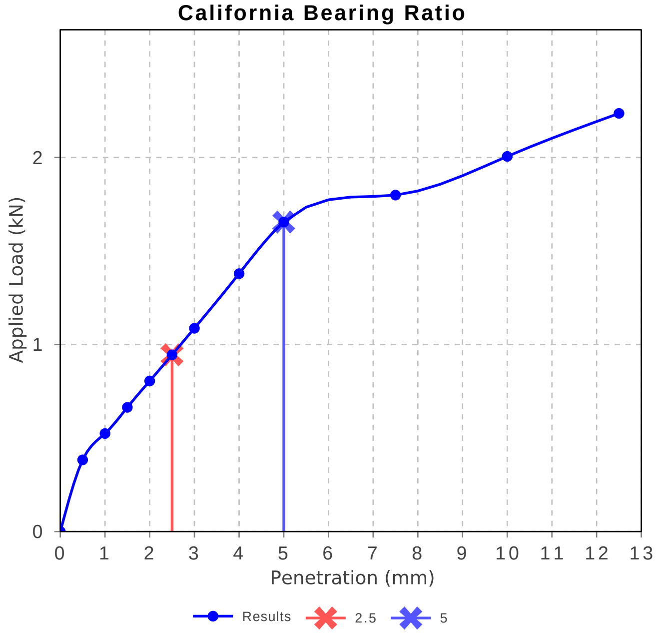
Douglas Partners
Geotechnics / Environment / Groundwater
Douglas Partners Pty Ltd
Coffs Harbour Laboratory
18 Lawson Crescent Coffs Harbour NSW 2450
Phone: (02) 6650 3200
Email: Brandon.Cameron@douglaspartners.com.au

Accredited for compliance with ISO/IEC 17025 - Testing

Approved Signatory: Brandon Cameron
Assistant Laboratory Manager
Laboratory Accreditation Number: 828

Report Number:	**209696.00-1**
Issue Number:	1
Date Issued:	16/03/2022
Client:	Northrop Consulting Engineers Pty Ltd Level 1, 215 Pacific Highway, Charlestown NSW 2290
Contact:	Ross Jeans
Project Number:	209696.00
Project Name:	Proposed Community Centres
Project Location:	River Street, Maclean & Treelands Drive, Yamba NSW
Work Request:	16025
Sample Number:	CF-16025A
Date Sampled:	21/02/2022
Dates Tested:	09/03/2022 - 15/03/2022
Sample Location:	**Bore 4 (0.1-0.4m)**
Material:	Dark Grey Silty Sand Fill

California Bearing Ratio (AS 1289 6.1.1 & 2.1.1)		Min	Max
CBR taken at	5 mm		
CBR %	**8**		
Method of Compactive Effort	Vibrated		
Method used to Determine MDD	N/A		
Method used to Determine Plasticity	Visual Assessment		
Maximum Dry Density (t/m^3)			
Optimum Moisture Content (%)			
Laboratory Density Ratio (%)			
Laboratory Moisture Ratio (%)			
Dry Density after Soaking (t/m^3)			
Field Moisture Content (%)	5.2		
Moisture Content at Placement (%)			
Moisture Content Top 30mm (%)	19.7		
Moisture Content Rest of Sample (%)	21.2		
Mass Surcharge (kg)	4.5		
Soaking Period (days)	4		
Curing Hours	2.4		
Swell (%)	-1.0		
Oversize Material (mm)	19		
Oversize Material Included	Excluded		
Oversize Material (%)	0		



Material Test Report

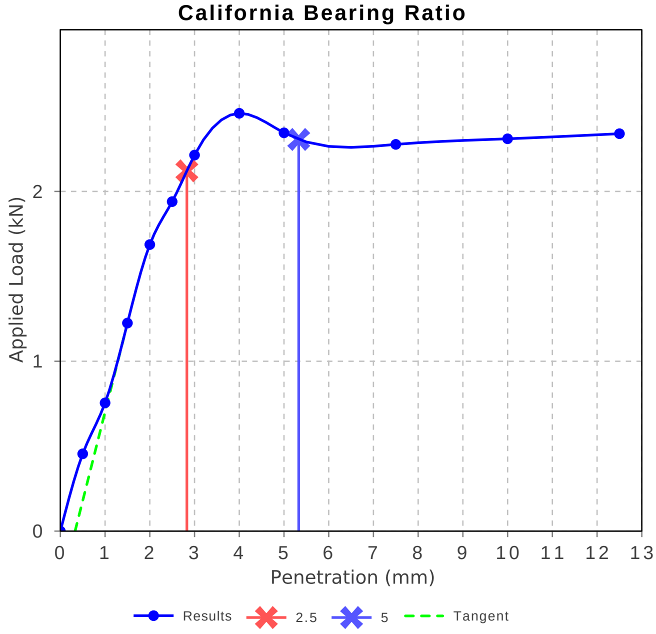
Douglas Partners
Geotechnics / Environment / Groundwater
Douglas Partners Pty Ltd
Coffs Harbour Laboratory
18 Lawson Crescent Coffs Harbour NSW 2450
Phone: (02) 6650 3200
Email: Brandon.Cameron@douglaspartners.com.au

Report Number:	**209696.00-1**
Issue Number:	1
Date Issued:	16/03/2022
Client:	Northrop Consulting Engineers Pty Ltd Level 1, 215 Pacific Highway, Charlestown NSW 2290
Contact:	Ross Jeans
Project Number:	209696.00
Project Name:	Proposed Community Centres
Project Location:	River Street, Maclean & Treelands Drive, Yamba NSW
Work Request:	16025
Sample Number:	CF-16025B
Date Sampled:	21/02/2022
Dates Tested:	09/03/2022 - 15/03/2022
Sample Location:	**Bore 5 (0.1-0.4m)**
Material:	Dark Grey Silty Sand Fill

Accredited for compliance with ISO/IEC 17025 - Testing

Approved Signatory: Brandon Cameron
Assistant Laboratory Manager
Laboratory Accreditation Number: 828

California Bearing Ratio (AS 1289 6.1.1 & 2.1.1)		Min	Max
CBR taken at	2.5 mm		
CBR %	**16**		
Method of Compactive Effort	Vibrated		
Method used to Determine MDD	N/A		
Method used to Determine Plasticity	Visual Assessment		
Maximum Dry Density (t/m^3)			
Optimum Moisture Content (%)			
Laboratory Density Ratio (%)			
Laboratory Moisture Ratio (%)			
Dry Density after Soaking (t/m^3)			
Field Moisture Content (%)	5.4		
Moisture Content at Placement (%)			
Moisture Content Top 30mm (%)	19.9		
Moisture Content Rest of Sample (%)	20.1		
Mass Surcharge (kg)	4.5		
Soaking Period (days)	4		
Curing Hours	2.6		
Swell (%)	0.0		
Oversize Material (mm)	19		
Oversize Material Included	Excluded		
Oversize Material (%)	0		



RESULTS OF ACID SULFATE SOIL ANALYSIS

8 samples supplied by Douglas Partners Pty Ltd on 24/02/2022. Lab Job No. M6482.
Analysis requested by Ryan Ussher. Your Job: 209696.

18 Lawson Cresent COFFS HARBOUR NSW 2450

Sample Identification	EAL Lab Code	Texture	Moisture Content		pH_F and pH_{FOX}			
			(% moisture of total wet weight)	(g moisture / g of oven dry soil)	pH_F	pH_{FOX}	pH change	Reaction
Method Info.		**	**		*(In-house method S21)*			
BH4 0.5	*M6482/1*	Coarse	2.3	0.02	8.27	6.27	-2.00	Medium
BH4 1.0	*M6482/2*	Coarse	3.3	0.03	9.21	6.60	-2.61	Low
BH4 1.5	*M6482/3*	Coarse	3.9	0.04	9.16	6.48	-2.68	Low
BH4 2.0	*M6482/4*	Coarse	10.4	0.12	8.90	6.44	-2.46	Low
BH5 0.5	*M6482/5*	Coarse	3.3	0.03	9.20	6.52	-2.68	Low
BH5 1.0	*M6482/6*	Coarse	7.7	0.08	8.15	5.80	-2.35	High
BH5 1.5	*M6482/7*	Coarse	4.9	0.05	8.74	6.29	-2.45	Medium
BH5 2.0	*M6482/8*	Coarse	12.8	0.15	8.25	5.95	-2.30	Medium

NOTES:

1. All analysis is reported on a dry weight (DW) basis, unless wet weight (WW) is specified.
2. Samples are dried and ground immediately upon arrival (unless supplied dried and ground).
3. Analytical procedures are sourced from Sullivan L, Ward N, Toppler N and Lancaster G. 2018. National acid sulfate soils guidance: national acid sulfate soils identification and laboratory methods manual, Department of Agriculture and Water Resources, Canberra, ACT. CC BY 4.0.
4. The Acid Base Accounting Equation, where Acid Neutralising Capacity has not been corroborated by other data, is Net Acidity = Potential Acidity + Actual Acidity + Retained Acidity (Eq. 3.2; Sullivan et al. 2018 - full reference above).
5. The Acid Base Accounting Equation for post-limed soil materials is Net Acidity = Potential Acidity + Actual Acidity + Retained Acidity - (post treatment Acid Neutralising Capacity - initial Acid Neutralising Capacity) (Eq. 3.3; Sullivan et al. 2018 - full reference above).
   While the Acid Neutralising Capacity of a soil material may not be included in the Net Acidity calculation (Note 4), it must be measured to give an Initial Acid Neutralising Capacity if verification testing is planned post-liming.
   The Inital Acid Neutralising Capacity must be provided by the client to enable EAL to produce Verification Net Acidity and Liming calculations for post-limed soil materials.
6. The Acid Base Accounting Equation, where Acid Neutralising Capacity has been corroborated by other data, is Net Acidity = Potential Acidity + Actual Acidity + Retained Acidity - Acid Neutralising Capacity (Eq. 3.1; Sullivan et al. 2018 - full reference above).
7. The lime calculation includes a Safety Factor of 1.5 as a safety margin for acid neutralisation (Sullivan et al. 2018). This is only applied to positive values. An increased Safety Factor may be required in some cases.
8. Retained Acidity is required when the pHKCl < 4.5 or where jarosite has been visually observed.
9. A negative Net Acidity result indicates an excess acid neutralising capacity.
10. If insufficient mixing occurs during intial sampling, or during post-liming, or both: the Potential Sulfidic Acidity may be greater in the post-limed sample than in the intial sample; the post-liming Acid Neutralising Capacity may be lower in the post-limed sample than in the intial sample.
11. **An acid sulfate soil management plan is triggered by Net Acidity results greater than the texture dependent criterion: coarse texture ≥ 0.03% S or 18 mol H+/t; medium texture ≥ 0.06% S or 36 mol H+/t; fine texture ≥ 0.1% S or 62 mol H+/t) (Table 1.1; Sullivan et al. 2018 - full reference above)**
12. For projects that disturb > 1000 t of soil material, the coarse trigger of ≥ 0.03% S or ≥ 18 mol H+/t must be applied in accordance with Sullivan et al. (2018) (full reference above).
13. Acid sulfate soil texture triggers can be related to NCST (2009) textures: coarse and peats = sands to loamy sands; medium = clayey sand to light clays; fine = light medium to heavy clays (Sullivan et al. 2018 - full reference above).
14. Bulk density is required to convert liming rates to soil volume based results. Field bulk density rings can be submitted to EAL for bulk density determination.
15. A negative Net Acidity result indicates an excess acid neutralising capacity.
16. '..' is reported where a test is either not requested or not required. Where pHKCl is < 4.5 or > 6.5, zero is reported for SNAS and ANC in Net Acidity calculations, respectively.
17. Results refer to samples as received at the laboratory. This report is not to be reproduced except in full.
18. ** NATA accreditation does not cover the performance of this service.
19. Analysis conducted between sample arrival date and reporting date.
20. All services undertaken by EAL are covered by the EAL Laboratory Services Terms and Conditions (refer SCU.edu.au/eal/t&cs or on request).
21. Results relate to the samples tested.
22. This report was issued on 08/03/2022.

NATA
WORLD RECOGNISED ACCREDITATION
Accreditation No. 14960
Accredited for compliance with ISO/IEC 17025 - Testing

checked: ....
Graham Lancaster
Laboratory Manager