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Integrated Practical Solutions

Report on Geotechnical Investigation

Proposed Light Towers
120 South Creek Road, Cromer

Prepared for
Northern Beaches Council

Project 214270.00
July 2022

Douglas Partners
Geotechnics | Environment | Groundwater

Document History

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Revision 0	1	0	Eliza Halsey, Northern Beaches Council

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

	Signature	Date
Author		11 July 2022
Reviewer		11 July 2022

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Geotechnics / Environment / Groundwater

Table of Contents

Report on Geotechnical Investigation
Proposed Light Towers
120 South Creek Road, Cromer

1. Introduction

This report presents the results of a geotechnical investigation undertaken for proposed light towers to be installed at Cromer High Sports Field, 120 South Creek Road, Cromer. The investigation was commissioned by Eliza Halsey of Northern Beaches Council and was undertaken in accordance with Douglas Partners' proposal 214270.00.P.001.Rev0 dated 8 March 2022.

It is understood that the proposed development of the site includes the installation of four light poles, each 20 m in height, at the perimeter of Cromer High sports field. Investigation is required to provide information on subsurface conditions and to provide parameters to assist in the design and planning of foundations.

The investigation included four Cone Penetration Tests (CPTu) using a piezocone to refusal and two hand augers. The details of the field work are presented in this report, together with comments and recommendations for design and construction.

2. Site Description

The site is a rectangular shaped area of approximately 120 m by 90 m in plan dimensions, within the grounds of Cromer High School. The site is bounded by the campus basketball courts to the north, South Creek Road to the south, Inman Road to the east and school buildings to the west. A location plan showing the site area and surrounds is presented in Figure 1 (following page).

At the time of the investigation, the site was used as a sports field. Facilities include a single level brick amenities building and underground storage tanks along the eastern section of the site. The remainder of the site is open grassed sports playing fields, and trees aligning with the eastern and southern boundary. Building infrastructure is generally of masonry construction, all of which was observed to be in good condition.

Topographically, the site is situated in a low-lying area which grades very gently to the south. Topographical relief across the oval is slight. The NSW 2 m elevation contour data from the NSW Department of Lands indicates a ground surface level of approximately RL 14 m AHD across most of the site and RL 12 m along the southern boundary.

Figure 1: Metro Map aerial image showing location of site

3. Regional Geology and Mapping

3.1 Geology

Reference to the Sydney 1:100,000 Geology Sheet (Wilson, 1983) indicates that the site is underlain by Quaternary stream alluvial and estuarine sediments (Qha) which typically comprise silty to peaty quartz sand, silt, and clay. The northern end of the oval is close to the boundary with Newport Formation rocks which comprises interbedded laminite, shale and quartz, to lithic-quartz sandstone. The fieldwork results were consistent with the broadscale geological mapping with alluvial sand encountered below the near-surface filling in Bores HA1 and HA3, and inferred to depths of 0.5 – 0.9 m in the CPTs.

3.2 Hydrogeology

Reference to data from Water NSW on historic groundwater bores indicates that there is a registered groundwater bore (GW108314) within 100 m, generally south (downslope) of the subject site. Review of the data indicates that the bore was drilled to a depth of approximately 78 m, with a standing water level of 5 m.

3.3 Acid Sulfate Soils

Reference to the 1:25 000 Acid Sulphate Soils (ASS) Risk map (D.T., 1998) indicates that the site is located within an area of "*no known occurrence of ASS materials*".

4. Field Work

4.1 Field Work Methods

The fieldwork was conducted on 20 April 2022 in the presence of a geotechnical engineer from DP. The investigation included four piezocone penetration tests (CPT 1 – 4) and two boreholes (Bores HA1 and HA3).

The CPTs undertaken taken to refusal of the cone probe at depths in the range of 13.1 m to 21.7 m. In a cone penetration test (CPT) a ballasted truck-mounted test rig is used to push a 35 mm diameter instrumented cone-tipped probe into the soil with a hydraulic ram system. Continuous measurements are made of the pressure on the cone tip and the friction on a 135 mm long sleeve located immediately behind the cone. The piezocone test was carried out to determine pore pressures induced by the cone penetration. The cone tip resistance, friction readings and dynamic pore pressure readings are displayed during the test and stored for subsequent plotting of results and interpretation. Groundwater observations were made after completion of the CPTs and withdrawal of the rods.

Two boreholes (HA1 and HA3) were drilled using hand tools to termination depths of 1.1 m and 1.2 m respectively. Boreholes were logged by a geotechnical engineer from DP. Boreholes were backfilled with drilling spoil upon completion and re-instated with grassed topsoil.

The locations of the boreholes and CPTs are shown on Drawing 1 in Appendix B. The co-ordinates and surface levels at the test locations, relative to Australia Height Datum (m, AHD), were interpolated from the provided site survey plan and are therefore approximate only. The levels are shown on the borehole and CPT logs, given in Appendix C.

4.2 Field Work Results

Details of the conditions encountered in the tests are presented in Appendix C, together with notes explaining classification methods and defining descriptive terms used in the preparation of the logs.

The general subsurface profile encountered is summarised below:

- Fill – typically loose sand to depths of about 0.2 m to 0.9 m; underlain by
- Alluvial Soils – Typically variable strength alluvial sands and clays to depths of about 2.9 m to 4.8 m; underlain by
  - – Medium dense to dense sands and stiff to very stiff clays to depths of about 12 m to 12.6 m; over
  - – Very stiff to hard clay with interbedded dense to very dense sand layers, extending down to the depth of the investigation;
  - – All CPTs were terminated upon refusal of the cone probe approximate depths of 13 m to 22 m within the hard clay or very dense sand, which might be extremely weathered bedrock, but this was not proved.

Groundwater was observed within CPT1, CPT2 and CPT4 after the withdrawal of the rods at levels of 0.4 m, 1.0 m and 0.5 m respectively (in the range of RL 13 to 13.6 m, AHD). Groundwater levels are transient and will fluctuate with weather and may be expected to rise by 1-2 m above the measured levels during periods following heavy rainfall. Extreme care must be exercises in the interpretation of ground water levels from coneholes due to the small (35 mm) diameter of the hole following cone probe withdrawal.

5. Geotechnical Model

Based on the results of the investigations an interpreted geotechnical model for the site comprises:

- Loose sand filling to depths of about 0.2 m to 0.9 m;
- Alluvial sands and clays of variable consistency to depths of about 2.9 m to 4.8 m;
- Medium dense to dense sand and stiff to very stiff very stiff clays to 12 to 12.6 m; over,
- Very stiff to hard clay with interbedded dense to very dense sand layers to approximately 13 m to 22 m, the depth of the investigation.

The CPTs refused at depths of approximately 13 m to 22 m which can occur on hard clays, very dense sands, or low strength rock; hence the material type below refusal depth is uncertain. The depth to rock was not assessed during the current investigation. If piles founded on rock are proposed then additional investigation would be required. Preliminary design parameters for rock have been given for preliminary design purposes only.

Groundwater was measured at depths of between 0.4 m and 1.0 m underlying the site (at about RL 13 m to RL 13.6 m AHD). It should be noted that groundwater levels are transient and will fluctuate with preceding climatic conditions.

6. Comments

6.1 Proposed Development

The proposed development of the site includes the installation of four new 20 m high lighting towers. The most suitable foundation system for the proposed light towers will depend on the axial and lateral design loads which have yet to be confirmed. No loads were provided at the time of preparing this report.

6.2 Excavation Conditions

Excavation will likely need to be carried out through fill and natural sands and clays which should be readily removed using conventional earthmoving equipment such as tracked hydraulic excavators.

Groundwater was measured at depths of between 0.4 m and 1 m on the site (at about RL 13 to 13.6 m AHD) which may lead to possibly unstable excavations if undertaken close to or beyond these depths/levels.

All excavated materials to be removed from site will need to be disposed of in accordance with the provisions of the current legislation and guidelines including the Waste Classification Guidelines (NSW EPA, 2014).

6.3 Foundations

The most suitable foundation system for the proposed light towers will depend on the axial and lateral design loads which have yet to be confirmed.

A range of pile types can be considered. Continuous flight auger (CFA) concrete injected piles could be considered for this site, as could concrete screw cast pile types such as Atlas or Omega piles. These pile types rely on the augers to keep the hole open during drilling and concrete is pumped down through the auger stem as it is withdrawn to complete the pile. Soil decompression can occur with these pile types when a strong stratum is encountered. This occurs when the augers continue to rotate but the rate of auger progression decreases, displacing soil from around the auger upwards towards the surface. Decompression can cause weakening and settlement of the soils adjacent to the pile and should be avoided by monitoring auger speed and progression closely. Open bored piles will not be appropriate due to the potential for soil collapse and groundwater ingress.

The use of steel screw piles could be adopted, however single screw piles may not have the capacity to resist lateral forces associated with wind loading, therefore a group of screw piles may be required. It is important that the installation of steel screw piles be carefully controlled by an experienced geotechnical engineer in the field to ensure the pile does not meet refusal prior to meeting its termination depth. The actual capacity of steel screw piles depends not only on the soil conditions but also on structural considerations of the piles such as the strength of the helix and the helix/shaft joint. As screw piles are a proprietary product, advice should be sought from specialist contractors with respect to capacities and founding depths.

For preliminary design of piles the parameters in Table 1 may be adopted.

Table 1: Preliminary Pile Design Parameters

Material Description	Unit Weight (kN/m^3)	Undrained Conditions		Maximum Allowable Parameters (Serviceability), kPa			Young's Modulus (E, MPa)
		Cohesion (C')	Friction Angle (^{0})	End Bearing[(3)]	Shaft Adhesion (Compression)	Shaft Adhesion (Tension)	
Filling	18	0	35	-	-	-	-
Loose sand	18	0	30	-	-	-	-
Medium dense sand	20	0	40	150	10	5	50

Material Description	Unit Weight (kN/m^3)	Undrained Conditions		Maximum Allowable Parameters (Serviceability), kPa			Young's Modulus (E, MPa)
		Cohesion (C')	Friction Angle (^{0})	End Bearing(3)	Shaft Adhesion (Compression)	Shaft Adhesion (Tension)	
Dense to very dense sand	20	0	40	300	20	10	100
Stiff to very stiff clays	20	35	0	150	25	12	15
Very stiff to hard clays	20	50	0	300	30	15	30
Weathered rock - assumed low strength(1)	20	-	-	700	70	35	50

Notes: (1) = depth to rock and strength of rock has not been proven by the investigations to date

(2)= allowable parameters generally associated with settlements of <1% of pile diameter

(3)= Bearing pressure values assume a minimum embedment of two pile diameters into the relevant bearing stratum, with an overall pile length of at least 5 m to extend the pile into the 'stable' zone, below the depth of seasonal moisture variation and 'shrink-swell' movements. If a weaker layer is present within four pile diameters below the pile toe then the end bearing capacity of the weaker layer should be adopted.

Shaft adhesion values should be reduced to 70% of the above values for the case of uplift (tension) loads and cone pull-out criteria should also be satisfied. Shaft adhesion is not applicable to steel screw piles.

An appropriate geotechnical strength reduction factor should be applied when using the limit-state approach as outlined in the piling code (AS 2159, 2009). Note that this would apply to 'ultimate' values, not the 'allowable' values given in Table 1. The determination of the geotechnical strength reduction factor (ϕ_g) uses a risk-based approach. For preliminary design purposes a factor of 0.4 could be assumed. Higher values of ϕ_g can be justified by more comprehensive static or dynamic load testing. The serviceability limit state should also be assessed in the design of the piles.

Footings must not be designed to terminate within the very soft to soft or soft to firm clay, or loose sands.

6.4 Seismic Design

Based on AS1170.4-2007 – "Structural Design actions Part 2: Earthquake actions in Australia" the following parameters could be adopted for seismic design:

- Seismic Hazard Factor (Z) 0.08; and
- Sub-soil Class, D_e.

7. References

AS 2159. (2009). *Piling - Design and Installation.* Standards Australia.

Chapman, G. M. (1989). Soil Landscapes Series - Sydney 1:100,000 Sheet. Soil Conservation Service of NSW.

D.T., N. S. (1998). *Guidelines for Use of Acid Sulfate Soils Risk Maps .* NSW Department of Land and Water Conservation.

PC. (2013). *Geotechncial Risk Management Policy for Pittwater - 2009.* No 178, Amended 4 November 2013: Pittwater Council (now Northern Beaches Council).

Sullivan, L., Ward, N., Toppler, N., & Lancaster, G. (2018). *National Acid Sulfate Soils Guidance: National Acid Sulfate Soils Sampling and Identification Methods Manual.* Canberra ACT CC BY 4.0: Department of Agriculture and Water Resources.

Walker, B. D. (2007). Practice Note guidelines for landslide risk management. *Australian Geomechanics Journal, Volume 42, Number 1*.

wilson. (n.d.).

Wilson, G. M. (1983). *Sydney 1:100,000 Geology Sheet.* NSW, Australia: NSW Department of Mines.

8. Limitations

Douglas Partners Pty Ltd (DP) has prepared this report for this project at 120 South Creek Road, Cromer in accordance with DP's proposal dated 8 March 2022 and acceptance received from Eliza Halsey of Northern Beaches Council. The work was carried out under DP and Northern Beaches Council consultancy services panel 2019/088 agreement. This report is provided for the exclusive use of Northern Beaches Council 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 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.

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Appendix A

About This Report

About this Report

Douglas Partners

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

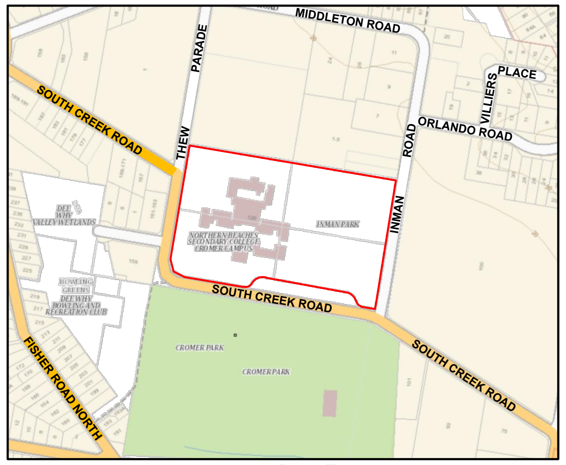
Where information obtained from this report is provided for tendering purposes, it is recommended that all information, including the written report and discussion, be made available. In circumstances where the discussion or comments section is not relevant to the contractual situation, it may be appropriate 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.

Appendix B

Drawing



Locality Plan

NOTE:
1: Base image from MetroMap (Dated 22.03.2022)

0 5 10 15 20 30 40 60 80m
1:800 @ A3

LEGEND

- Approximate Site Boundary
- Approximate Hand Auger/CPT Location
- Approximate CPT Location

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CLIENT: Northern Beaches Council		TITLE: **Test Location Plan**	PROJECT No: 214270.00
OFFICE: Sydney	DRAWN BY: MG	**Proposed Light Towers**	DRAWING No: 1
SCALE: 1:800 @ A3	DATE: 27.04.2022	**120 South Creek Road, Cromer**	REVISION: 0

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Appendix C

Results of the Investigation

BOREHOLE LOG

CLIENT: Northern Beaches Council
PROJECT: Proposed Light Towers
LOCATION: 120 South Creek Road, Cromer

SURFACE LEVEL: 14.0 AHD
EASTING: 341128.8
NORTHING: 6265611.5
DIP/AZIMUTH: 90°/--

BORE No: HA1
PROJECT No: 214270.00
DATE: 20/4/2022
SHEET 1 OF 1

RL	Depth (m)	Description of Strata	Graphic Log	Sampling & In Situ Testing: Type	Depth	Sample	Results & Comments	Water	Well Construction Details
14		FILL/TOPSOIL/Silty SAND: fine to medium, dark brown, non-plastic fines, trace fine gravel, with rootlets, moist, apparently in a loose condition		D/E	0.1–0.2				
	0.2	FILL/Clayey SAND: fine to medium, pale brown, low plasticity clay, moist, apparently in a loose condition		D/E	0.3–0.4				
	0.4	Clayey SAND SC: fine to medium, orange-brown, low plasticity clay, wet, loose, alluvium		D/E	0.5–0.6				
13	1								1
	1.1	Bore discontinued at 1.1m							

RIG: Hand Tools **DRILLER:** RAS **LOGGED:** RAS **CASING:** Uncased
TYPE OF BORING: 100 mm Hand Auger to 1.1m
WATER OBSERVATIONS: Standing Water Level encountered at 0.4m
REMARKS: Location coordinates are in MGA94 Zone 56.

SAMPLING & IN SITU TESTING LEGEND

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

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BOREHOLE LOG

CLIENT: Northern Beaches Council
PROJECT: Proposed Light Towers
LOCATION: 120 South Creek Road, Cromer

SURFACE LEVEL: 14.0 AHD
EASTING: 341196.5
NORTHING: 6265542.9
DIP/AZIMUTH: 90°/--

BORE No: HA3
PROJECT No: 214270.00
DATE: 20/4/2022
SHEET 1 OF 1

RL	Depth (m)	Description of Strata	Graphic Log	Sampling & In Situ Testing: Type	Depth	Sample	Results & Comments	Water	Well Construction Details
14		FILL/TOPSOIL/Silty SAND: fine to medium, dark brown, non-plastic fines, trace fine gravel, moist, apparently in a loose condition		D/E	0.1–0.2				
	0.2	SAND SP: fine to medium, pale grey, trace silt, moist, loose, alluvium		D/E	0.5–0.6				
		Below 0.8m: becoming pale brown							
13	1			D/E	1.0–1.2				1
	1.2	Bore discontinued at 1.2m							

RIG: Hand Tools **DRILLER:** RAS **LOGGED:** RAS **CASING:** Uncased
TYPE OF BORING: 100 mm Hand Auger to 1.2m
WATER OBSERVATIONS: No free groundwater observed whilst augering
REMARKS: Location coordinates are in MGA94 Zone 56.

SAMPLING & IN SITU TESTING LEGEND

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

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CONE PENETRATION TEST

CLIENT: NORTHERN BEACHES COUNCIL

PROJECT: PROPOSED LIGHT TOWERS

LOCATION: 120 SOUTH CREEK ROAD, CROMER

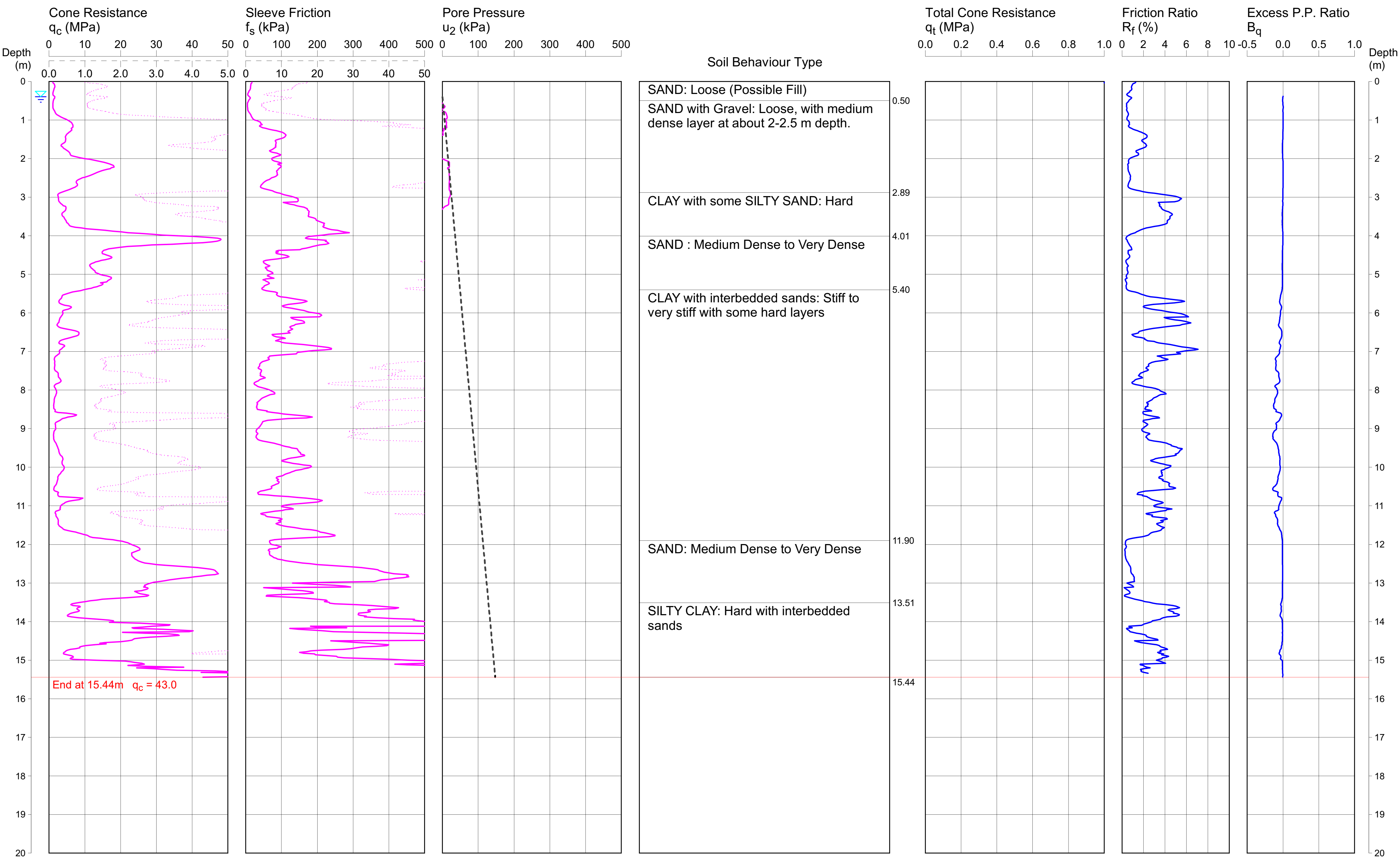
REDUCED LEVEL: 14.0

COORDINATES: 341128.8E 6265611.5N AHD

CPT1

DATE 20/04/2022

PROJECT No: 214270



Depth (m)	Soil Behaviour Type
0.00 – 0.50	SAND: Loose (Possible Fill)
0.50 – 2.89	SAND with Gravel: Loose, with medium dense layer at about 2-2.5 m depth.
2.89 – 4.01	CLAY with some SILTY SAND: Hard
4.01 – 5.40	SAND : Medium Dense to Very Dense
5.40 – 11.90	CLAY with interbedded sands: Stiff to very stiff with some hard layers
11.90 – 13.51	SAND: Medium Dense to Very Dense
13.51 – 15.44	SILTY CLAY: Hard with interbedded sands

REMARKS: TEST DISCONTINUED DUE TO EXCESSIVE BENDING.
GROUNDWATER OBSERVED AT 0.4M

Water depth after test: 0.40m depth (measured)

File: \\dpsydnas02\Projects\214270.00 - CROMER, 120 South Creek Road\4.0 Field Work\4.2 Testing\214270.00 - Cromer\CPT1.CP5
Cone ID: 210732 **Type:** I-CFXYP20-10

ConePlot Version 5.9.2

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CONE PENETRATION TEST

CLIENT: NORTHERN BEACHES COUNCIL

PROJECT: PROPOSED LIGHT TOWERS

LOCATION: 120 SOUTH CREEK ROAD, CROMER

REDUCED LEVEL: 14.0

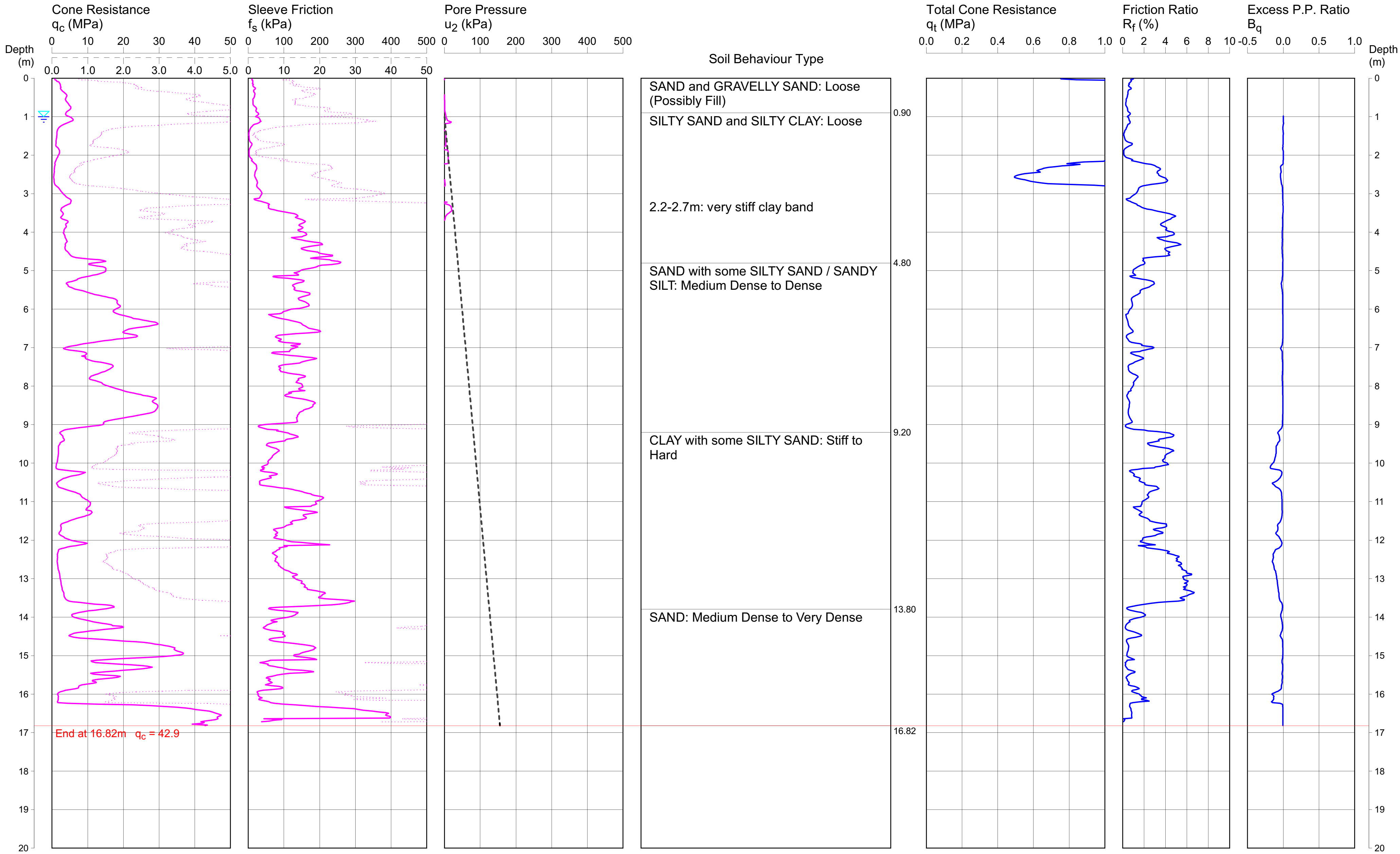
COORDINATES: 341121.0E 6265550.3N AHD

CPT2

Page 1 of 1

DATE 20/04/2022

PROJECT No: 214270



REMARKS: TEST DISCONTINUED DUE TO EXCESSIVE BENDING.
GROUNDWATER OBSERVED AT 1.0M

Water depth after test: 1.00m depth (measured)

File: \\dpsydnas02\Projects\214270.00 - CROMER, 120 South Creek Road\4.0 Field Work\4.2 Testing\214270.00 - Cromer\CPT2.CP5
Cone ID: 210410 **Type:** I-CFXYP20-10

ConePlot Version 5.9.2

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CONE PENETRATION TEST

CLIENT: NORTHERN BEACHES COUNCIL

PROJECT: PROPOSED LIGHT TOWERS

LOCATION: 120 SOUTH CREEK ROAD, CROMER

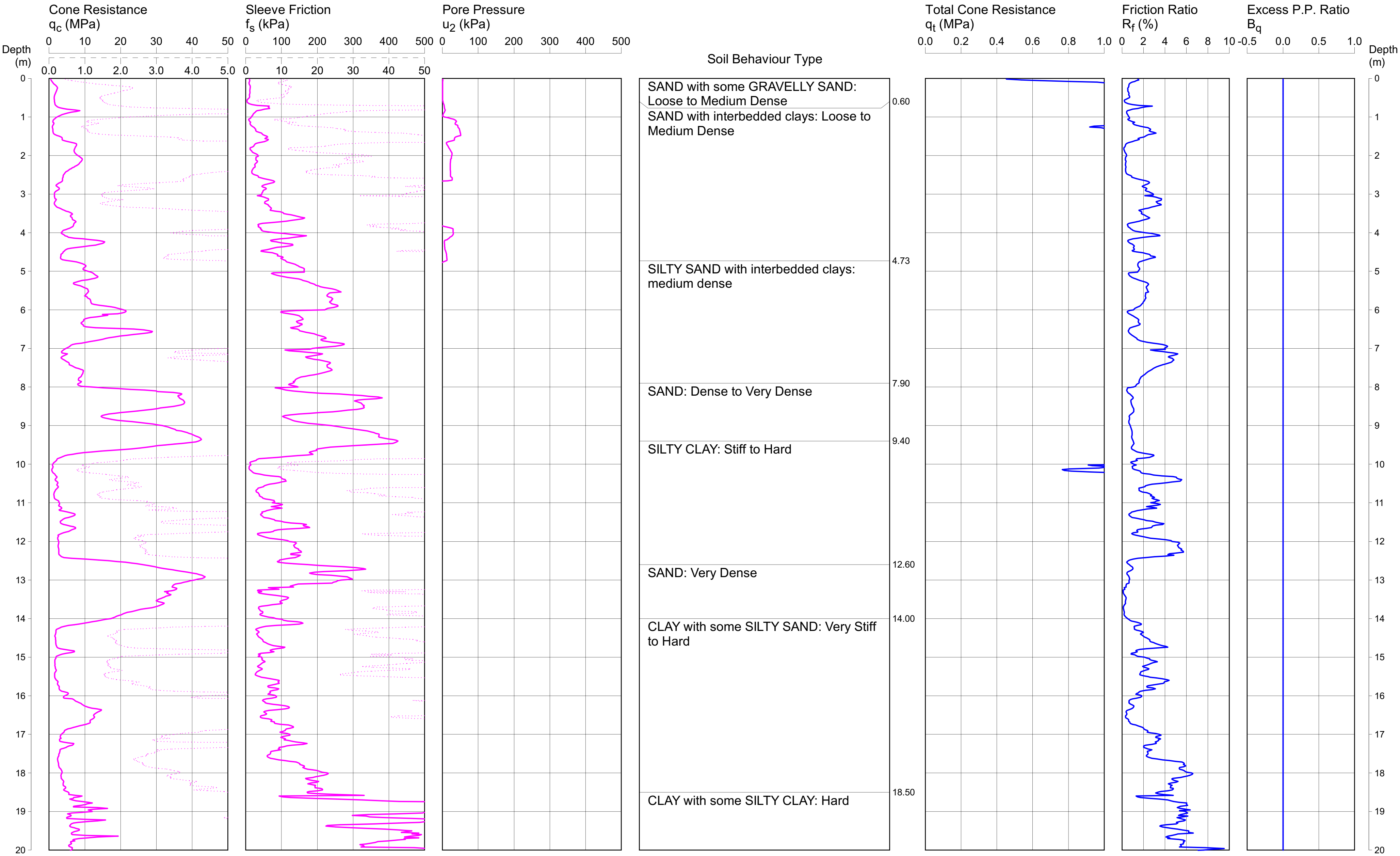
REDUCED LEVEL: 14.0

COORDINATES: 341196.5E 6265542.9N AHD

CPT3

DATE 20/04/2022

PROJECT No: 214270



REMARKS: TEST DISCONTINUED DUE TO EXCESSIVE BENDING.
STANDING WATER LEVEL COULD NOT BE MEASURED

File: \\dpsydnas02\Projects\214270.00 - CROMER, 120 South Creek Road\4.0 Field Work\4.2 Testing\214270.00 - Cromer\CPT3.CP5
Cone ID: 200309 **Type:** I-CFXYP20-10

ConePlot Version 5.9.2

Douglas Partners
Geotechnics / Environment / Groundwater

CONE PENETRATION TEST

CLIENT: NORTHERN BEACHES COUNCIL

PROJECT: PROPOSED LIGHT TOWERS

LOCATION: 120 SOUTH CREEK ROAD, CROMER

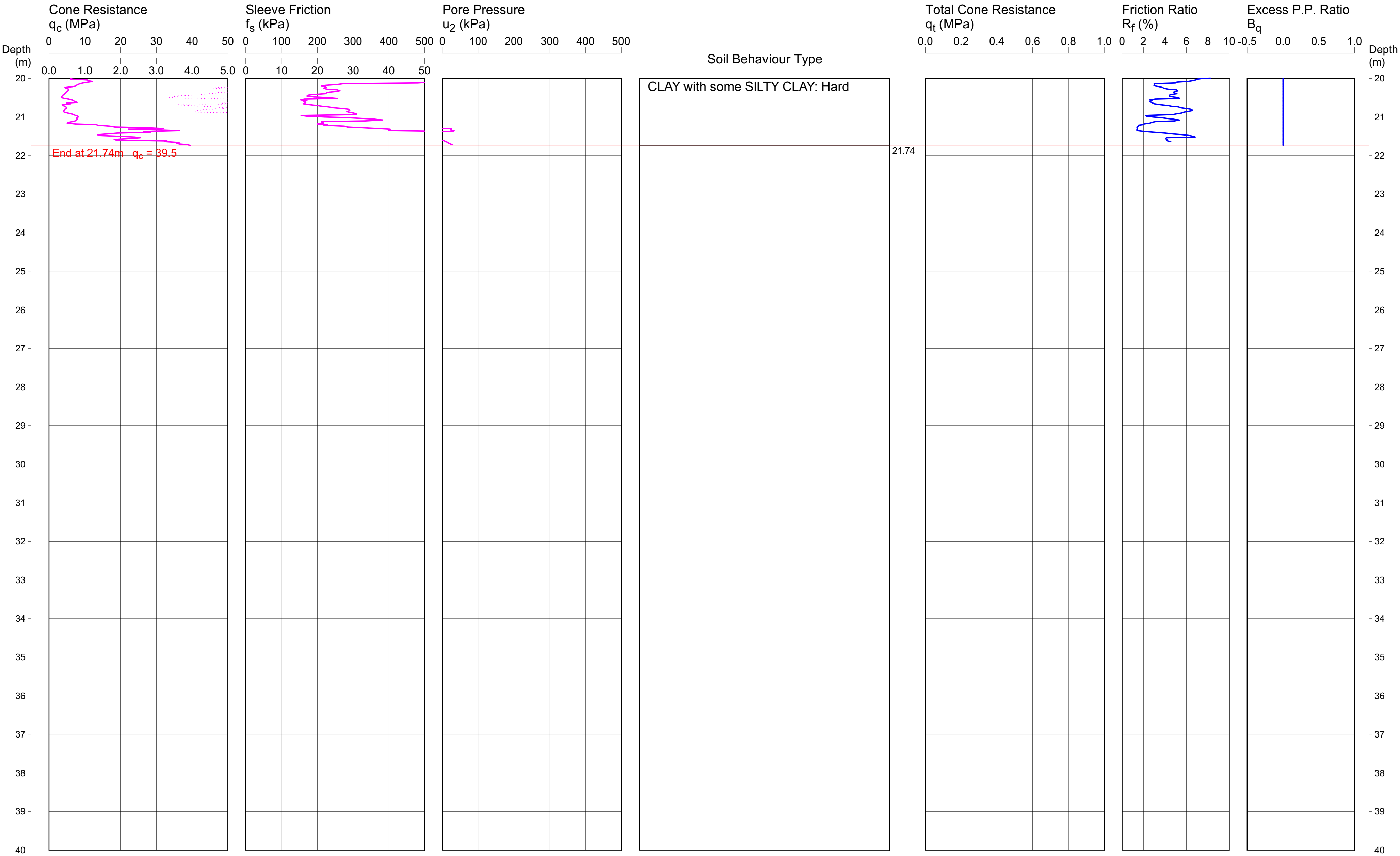
REDUCED LEVEL: 14.0

COORDINATES: 341196.5E 6265542.9N AHD

CPT3

DATE 20/04/2022

PROJECT No: 214270



REMARKS: TEST DISCONTINUED DUE TO EXCESSIVE BENDING.
STANDING WATER LEVEL COULD NOT BE MEASURED

File: \\dpsydnas02\Projects\214270.00 - CROMER, 120 South Creek Road\4.0 Field Work\4.2 Testing\214270.00 - Cromer\CPT3.CP5

Cone ID: 200309 **Type:** I-CFXYP20-10

ConePlot Version 5.9.2

Douglas Partners
Geotechnics / Environment / Groundwater

CONE PENETRATION TEST

CLIENT: NORTHERN BEACHES COUNCIL

PROJECT: PROPOSED LIGHT TOWERS

LOCATION: 120 SOUTH CREEK ROAD, CROMER

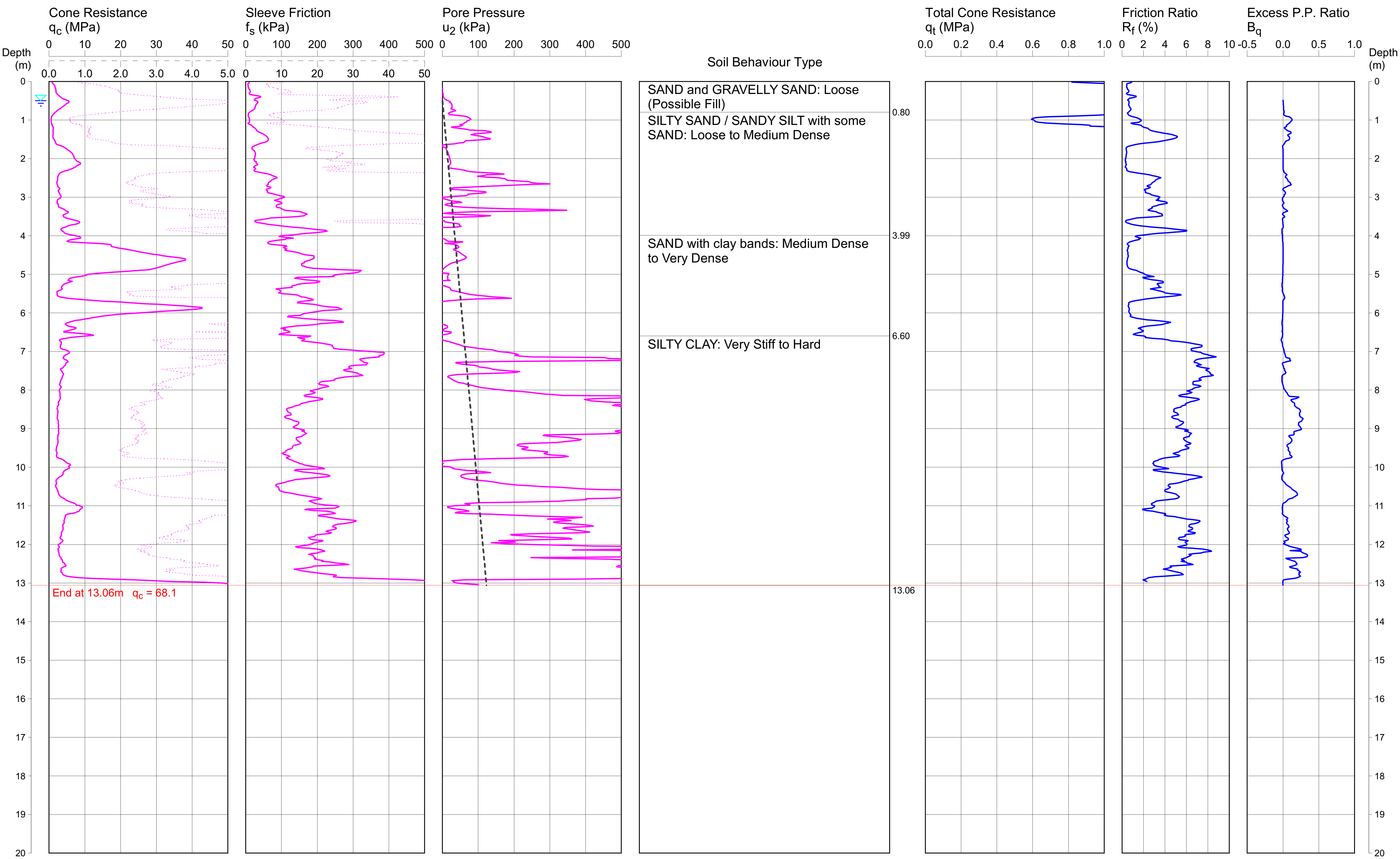
REDUCED LEVEL: 14.0

COORDINATES: 341205.6E 6265591N AHD

CPT4

DATE 20/04/2022

PROJECT No: 214270



REMARKS: TEST DISCONTINUED DUE TO EXCESSIVE BENDING. GROUNDWATER OBSERVED AT 0.5M

Water depth after test: 0.50m depth (measured)

File: \\dpsydnas02\Projects\214270.00 - CROMER, 120 South Creek Road\4.0 Field Work\4.2 Testing\214270.00 - Cromer\CPT4.CP5
Cone ID: 200310 **Type:** I-CFXYP20-10

ConePlot Version 5.9.2

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Geotechnics | Environment | Groundwater

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).

Rock Strength

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

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

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

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

Degree of Weathering

The degree of weathering of rock is classified as follows:

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

Rock Descriptions

Degree of Fracturing

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

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

Rock Quality Designation

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

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

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

Stratification Spacing

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

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

Symbols & Abbreviations

Douglas Partners

Introduction

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

Drilling or Excavation Methods

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

Water

▷	Water seep
▽	Water level

Sampling and Testing

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

Description of Defects in Rock

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

Defect Type

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

Orientation

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

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

Coating or Infilling Term

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

Coating Descriptor

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

Shape

cu	curved
ir	irregular
pl	planar
st	stepped
un	undulating

Roughness

po	polished
ro	rough
sl	slickensided
sm	smooth
vr	very rough

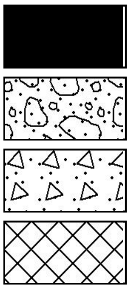
Other

fg	fragmented
bnd	band
qtz	quartz

Symbols & Abbreviations

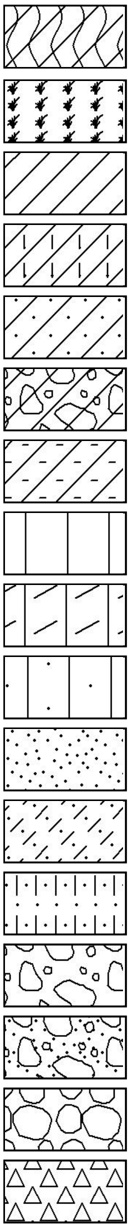
Graphic Symbols for Soil and Rock

General



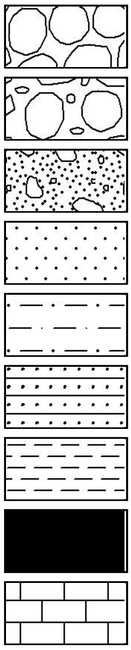
Asphalt
Road base
Concrete
Filling

Soils



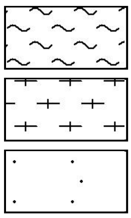
Topsoil
Peat
Clay
Silty clay
Sandy clay
Gravelly clay
Shaly clay
Silt
Clayey silt
Sandy silt
Sand
Clayey sand
Silty sand
Gravel
Sandy gravel
Cobbles, boulders
Talus

Sedimentary Rocks



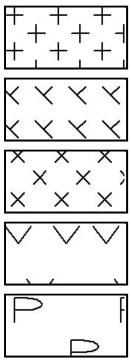
Boulder conglomerate
Conglomerate
Conglomeratic sandstone
Sandstone
Siltstone
Laminite
Mudstone, claystone, shale
Coal
Limestone

Metamorphic Rocks



Slate, phyllite, schist
Gneiss
Quartzite

Igneous Rocks



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

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

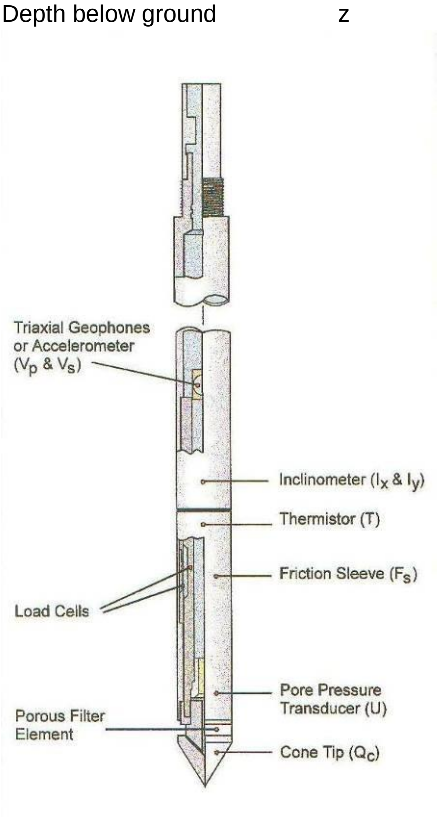


Figure 1: Cone Diagram

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)

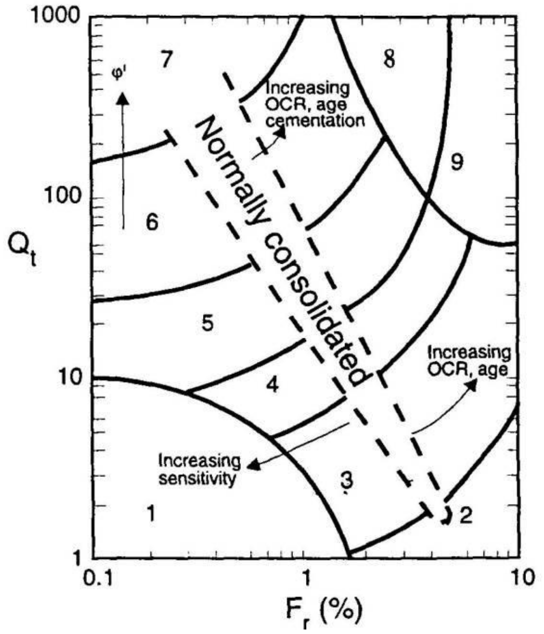


Figure 3: Soil Classification Chart

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.

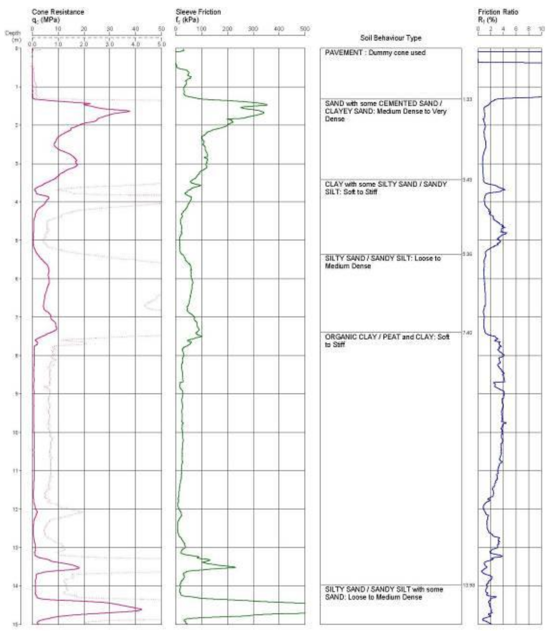


Figure 4: Sample Cone Plot