L7-17 Geotechnical Site Investigation

Investigation Results

Geotechnics

F225236/01 | 04 | 3 August 2023

Final

Neptune Energy Netherlands B.V.

Document Control

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Client	Neptune Energy Netherlands B.V.
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1. Introduction

1.1 Purpose and Scope of Report

Neptune Energy Netherlands B.V., henceforth referred to as 'client', is planning development of proposed well 'L7-17', in Block L7 of the Dutch sector of the North Sea. L7-17 is located approximately 70 km northwest of Den Helder, the Netherlands (refer to Plate A-1 'Vicinity Map' and Plate A-2 'Detailed Location Plan').

A project-specific geotechnical site investigation was performed by Fugro to acquire information in support of jack-up placement and foundation design for a piled production platform. The investigation included geotechnical borehole drilling with downhole sampling and in situ testing.

This document presents results of the geotechnical site investigation, and includes:

- One geotechnical log with results from downhole sampling and cone penetration testing to 100.7 m below seafloor (BSF);
- Results of downhole cone penetration testing;
- Results of geotechnical laboratory testing on a selection of samples.

1.2 Project Responsibilities and Use of Report

This report presents information according to a project specification determined and monitored by the client. The Main Text section titled 'Sources of Information and References' provides further details. Particularly, the client selected and approved test location and depths, monitored and approved data acquisition, and supplied information for geotechnical project use.

The report was prepared in accordance with contract P-A4364-00500-C02, between Neptune Energy Netherlands B.V. and Fugro Netherlands Marine B.V., dated 2 February 2023.

Read this report in its entirety. Particularly, take careful note of the document 'Use of Geodata and Advice', presented in the appendix 'Descriptions of Methods and Practices'.

This report uses and summarises information from sources listed in Section 3. The reader should consult the source information for details. Understanding of site conditions improves upon further data acquisition and interpretation. This means that some of the source interpretations may be superseded by information presented in this report.

Fugro understands that this report will be used for the purpose described in this Main Text section. That purpose was a significant factor in determining the scope and level of the services. Results must not be used if the purpose for which the report was prepared or the client's proposed development or activity changes. Results may possibly suit alternative use. Suitability must be verified.

2. Investigation Programme

2.1 Project-specific Data Acquisition

This report includes results from a project-specific geotechnical site investigation. The client planned the investigation programme. During the investigation, the client's programme was adjusted to suit as-found conditions and operational constraints. The scope of the report includes the results of the final programme.

The site phase of the geotechnical site investigation was conducted from geotechnical drilling vessel 'Gargano' between 26 and 28 February 2023. Water depth at the investigated location was approximately 25 m relative to lowest astronomical tide (LAT) (refer to plate A-3 'Coordinates and Water Depth').

Acquired data and recovered samples were processed in the onsite laboratory. Samples were stored onboard in a temperature-controlled environment and transferred to shore for additional laboratory testing and sample storage. An in-office phase comprising a laboratory testing programme and reporting followed the site phase.

The following sub-sections provide details.

2.2 Positioning Survey and Water Depth Measurement

Coordinates and water depths are presented on Plate A-3. The coordinate reference system uses European Datum 1950 (ED50) as datum with spheroid International 1924. The horizontal projection is Universal Transverse Mercator (UTM) with a Central Meridian of 3° east (Zone 31 N), refer to Plates A-4 and A-5 'Geodetic Parameters'.

Subsurface positioning was performed using an ultra short baseline (USBL) system with a transponder mounted on the seabed frame. Sound velocity profiles were obtained by lowering a conductivity, temperature, and depth (CTD) probe to seafloor.

LAT and seafloor are used as vertical reference levels for water depth measurement and geotechnical testing and sampling. Water depth measurements were performed using:

- a CTD probe, mounted on the seabed frame;
- a USBL system, with a transducer mounted on the hull of the vessel and a transponder located on the seabed frame;
- drill string by direct sounding when drilling was initiated for downhole sampling or downhole in situ testing.

The user of the geodetic information presented must consider the accuracy of measurements, particularly where use may differ from original intentions. For example, the presented water depth measurements serve to establish the depths below seafloor.

2.3 Drilling, Sampling and In Situ Testing

The site investigation consisted of drilling of one borehole to a depth of 100.7 m BSF with continuous downhole cone penetration testing (CPT) to a depth of 30.8 m BSF (including oversampling of the first 3 m BSF), followed by alternating CPTs and sampling.

The employed drilling method was open-hole rotary drilling. A combination of seawater and guar gum or Pure-Bore® was used as drill fluid. Borehole drilling included the use of a SEACLAM seabed frame to facilitate re-entry of the drill string in the borehole and for axial and lateral support of the drill string at seafloor.

Downhole push sampling and in situ testing employed WIPSAMPLER® and WISON® downhole tools. The sampler was fitted with flush stainless steel Shelby tubes, fitted with a core catcher where appropriate. Downhole tools were operated and retrieved by a hydraulic-electrical umbilical system which provides real-time control of the in situ testing and sampling process. Downhole CPTs were performed using cone penetrometers with a 1000 mm^2 (CP10) or 500 mm^2 (CP5) cone tip area.

Oversamples are samples recovered (partially) within the depth interval in which a previous in situ test has been performed. These samples have been acquired using the same procedure described for standard downhole push sampling.

Refer to the appendices titled 'Geotechnical Logs', 'Results of Cone Penetration Tests', and 'Geotechnical Laboratory Test Results' for details and results. Refer to the appendix titled 'Description of Methods and Practices' for details about methodology and test procedures.

2.4 Sample Handling

Onsite sample handling comprised the following:

- Removal of the sample tube from the sampling tool;
- Measurement of sample recovery in the sample tube;
- Transfer of the sample tube to the site laboratory;
- Sample extrusion, sample photography, and soil description and classification;
- Laboratory testing on selected sample sections;
- Sample packaging, labelling, and storage in a temperature-controlled container onboard;
- Sample offloading and transport to the Fugro geotechnical laboratory in Wallingford (UK) and other Fugro-nominated laboratories.

Office sample handling comprised the following:

- Sample delivery and registration at Fugro geotechnical laboratory in Wallingford and other Fugro-nominated laboratories;
- Selection of specimens for laboratory testing and preparation for testing;
- Laboratory testing (refer to Appendix E 'Geotechnical Laboratory Test Results' for details);

Further details on sample handling are provided in appendix titled 'Geotechnical Laboratory Test Results'.

2.5 Laboratory Testing

Onsite geotechnical laboratory testing comprised (where appropriate) visual soil description and classification of samples according to BS 5930:2015+A1:2020, ISO 14688-1:2017, and ISO 14688-2:2017, sample photography of extruded sample material, and geotechnical index.

Office geotechnical laboratory testing comprised (where appropriate) geotechnical index, geochemical, strength, and interface shear.

Laboratory test results are provided in the appendix titled 'Geotechnical Laboratory Test Results' and summaries of selected test results are presented on the geotechnical log in the appendix titled 'Geotechnical Log'.

2.6 Geotechnical Data Processing

Geotechnical data processing included:

- Preparation of a geotechnical log by interpreting and integrating sample descriptions and laboratory test results, where applicable;
- Evaluation, basic verification where practicable, and incorporation of selected information supplied by client.

Further details on geotechnical data processing are described in the appendices titled 'Geotechnical Logs', 'Results of Cone Penetration Tests', and 'Geotechnical Laboratory Test Results', where appropriate.

3. Sources of Information and References

3.1 Client-supplied Information

This report summarises and relies on a client-supplied scope of work (NENL, 2023).

3.2 Fugro Information

This report uses and summarises Fugro-held information:

- Information about regional geology;
- General geotechnical data.

3.3 References in Main Text

BSI. (2020). *Code of practice for ground investigations* (BS 5930:2015+A1:2020). British Standards Institution. https://knowledge.bsigroup.com/products/code-of-practice-for-ground-investigations/standard

ISO. (2017). *Geotechnical investigation and testing - Identification and classification of soil - Part 1: Identification and description* (ISO 14688-1:2017). International Organization for Standardization. https://www.iso.org/standard/66345.html

ISO. (2017). *Geotechnical investigation and testing - Identification and classification of soil - Part 2: Principles for a classification* (ISO 14688-2:2017). International Organization for Standardization. https://www.iso.org/standard/66346.html

NENL. (2023). *Geotechnical Survey Location L7-17. Dutch Continental Shelf - Client Technical Specifications* (Document No. NENL-GEN-CTS-013), rev. 5. Neptune Energy Netherlands B.V.

Appendix A

Plates referenced in Main Text

Contents Appendix A: Plates referenced in Main Text

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Coordinates and Water Depth	A-3
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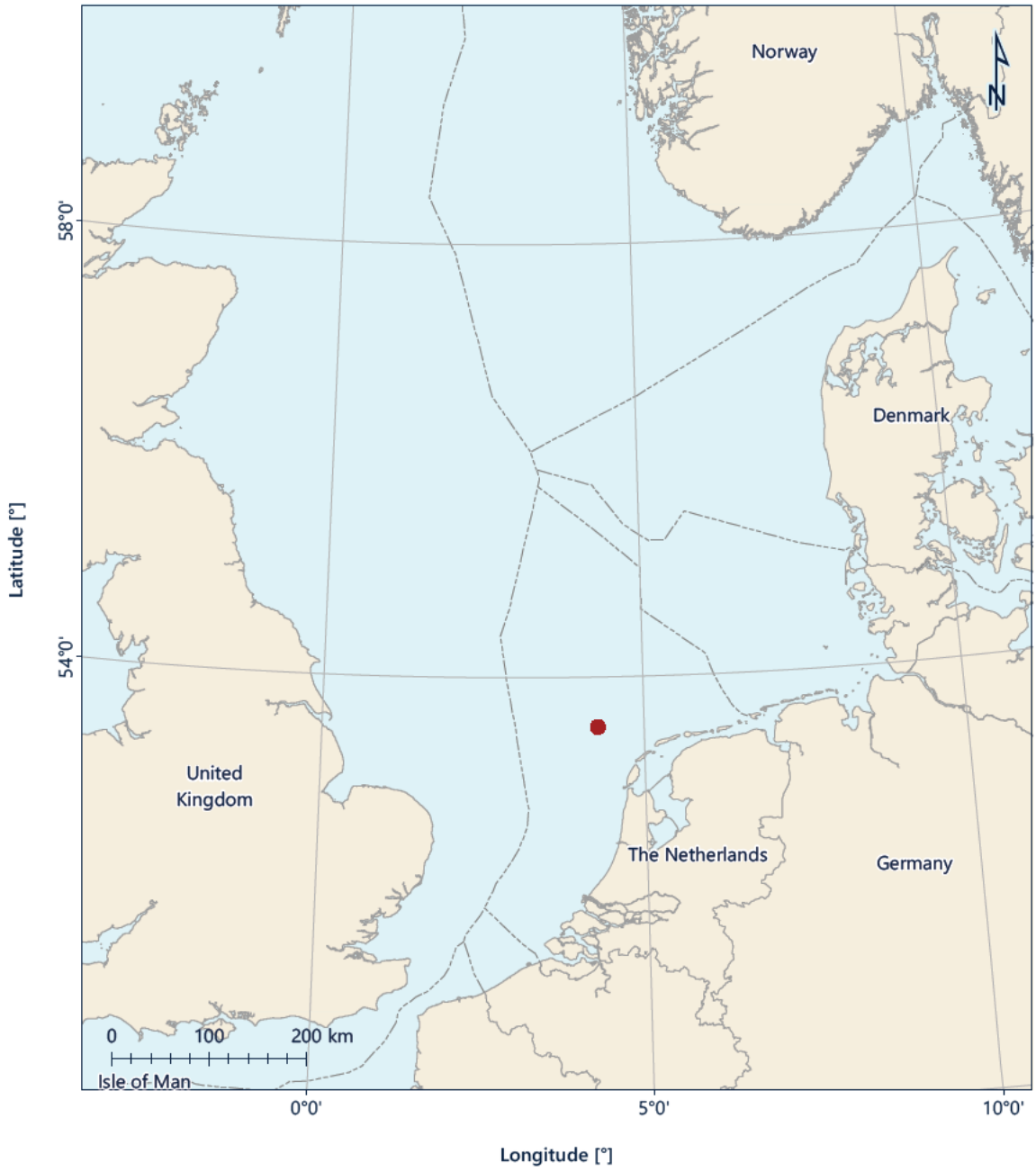
Practice for Positioning Survey and Depth Measurement	A-6 to A-7



Approximate area of investigation
Maritime boundary

Datum : European Datum 1950
Ellipsoid : International 1924
Projection : Universal Transverse Mercator (UTM)
Central Meridian : 3° E (UTM Zone 31N)

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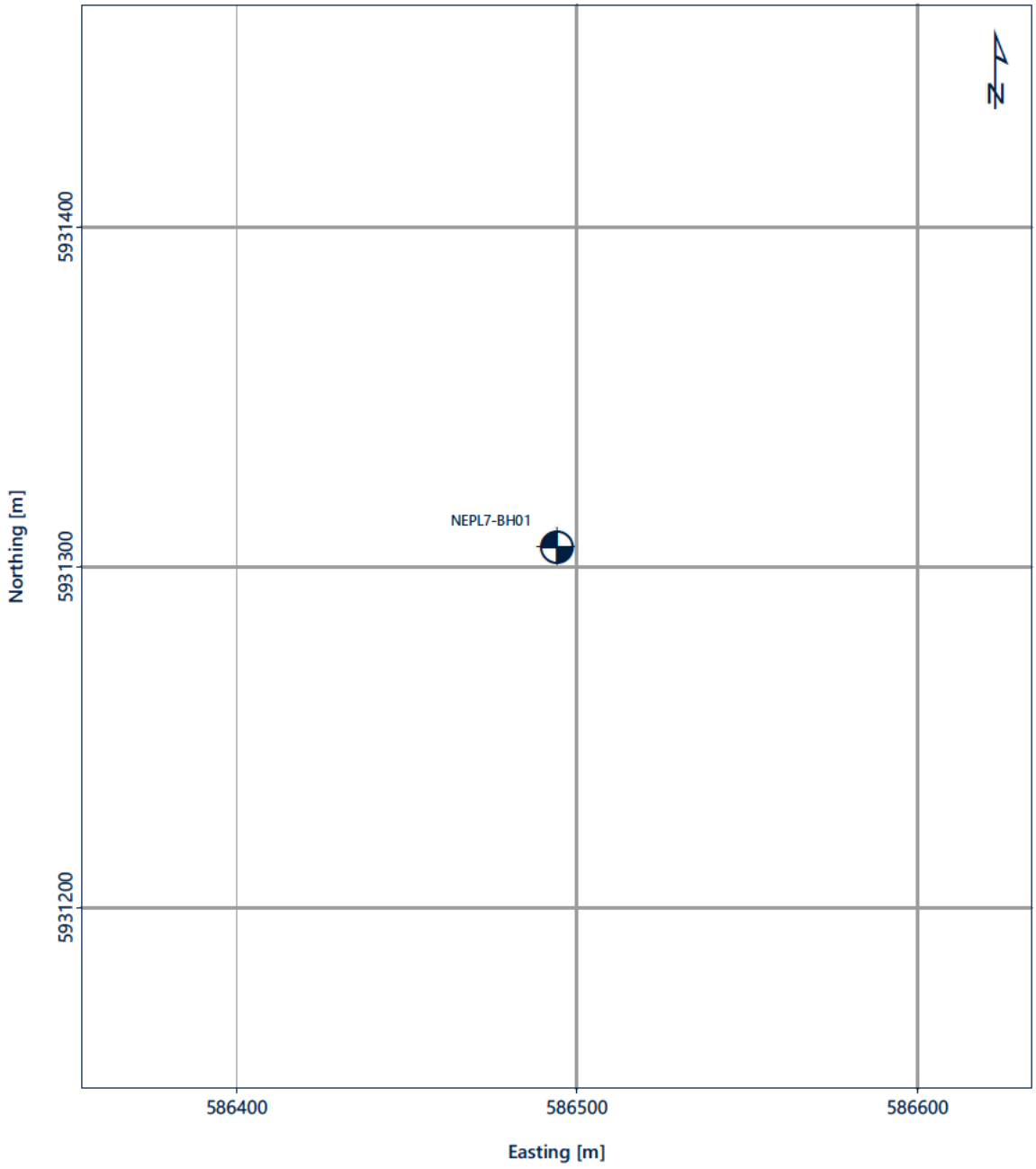
Vicinity Map



Geotechnical locations

Rotary borehole drilling, sampling and testing

Datum	: European Datum 1950
Ellipsoid	: International 1924
Projection	: Universal Transverse Mercator
Central Meridian	: 3° E (UTM Zone 31 N)

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Detailed Location Plan

COORDINATES

Borehole	Penetration Depth [m]	Easting [m]	Northing [m]	Latitude [deg]	Longitude [deg]
NEPL7-BH01	100.7	586494	5931306	53°31'20.7" N	4°18'16.6" E

Datum	: ED 50	Projection	: Universal Transverse Mercator
Ellipsoid	: International 1924	Central Meridian	: 3° E (UTM Zone 31N)

Penetration Depth: deepest point reached by drilling, in situ testing or sampling, relative to seafloor

WATER DEPTH

Borehole	Date	CTD Probe Reduced [m]	USBL Reduced [m]	Drill String Reduced [m]
NEPL7-BH01	28-Feb-2023	25.4	25.7	25.6

Reduced water depths are relative to Lowest Astronomical Tide (LAT)

GeODin/Coordinates and Water Depths.GLO/2023-03-06 10:57:05

Project Coordinate Reference System	
Coordinate Reference System	ED50/UTM zone 31N
EPSG Code	23031
Global Geodetic Parameters	
Coordinate Reference System	World Geodetic System 1984 (WGS 84)
Datum	WGS 84
Ellipsoid	WGS 84
Semi-Major Axis, a	6 378 137.00 m
Inverse Flattening, $1/f$	298.257223563
EPSG Code (Coordinate Reference System)	4326
EPSG Code (Datum)	6326
EPSG Code (Ellipsoid)	7030
Local Geodetic Parameters	
Coordinate Reference System	European Datum 1950 (ED50)
Datum	ED50
Ellipsoid	International 1924
Semi-Major Axis, a	6 378 388.00 m
Inverse Flattening, $1/f$	297.0000000
EPSG Code (Coordinate Reference System)	4230
EPSG Code (Datum)	6230
EPSG Code (Ellipsoid)	7022
Transformation Parameters	
Transformation Name	WGS84 to ED50 (UKOOA)
Shift dX	+89.5 m
Shift dY	+93.8 m
Shift dZ	+123.1 m
Rotation rX	0
Rotation rY	0
Rotation rZ	-0.156"
Scale Factor	-1.200 ppm
EPSG Code (Transformation)	1311
Projection Parameters	
Projection Method	Universal Transverse Mercator
Projection Name	UTM zone 31N
Latitude of Natural Origin	00° 00' 00.0000" N
Longitude of Natural Origin (Central Meridian)	03° 00' 00.0000" E
False Easting	500 000 m
False Northing	0 m
Scale Factor at Natural Origin	0.9996
Unit	Metre
EPSG Code (Projection)	16031

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Geodetic Parameters

Example Point		
Local Projected Coordinates	Easting	608 893.3 m
	Northing	6 079 919.5 m
Local Geographic Coordinates	Latitude	54° 51' 11.961"N
	Longitude	04° 41' 46.007"E
Global Geographic Coordinates	Latitude	54° 51' 0.9.445"N
	Longitude	04° 41' 41.022"E

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Overview

General Procedure:	■ According to ISO (2014) ■ Metrological confirmation according to ISO (2003) ■ Refer to document titled 'Positioning Survey and Depth Measurement' presented in appendix titled 'Descriptions of Methods and Practices'
Purpose of Measurement(s):	■ Provision of spatial (xyz) position of data points applicable to geotechnical investigation location(s) ■ The user of the presented positioning survey and depth measurement data must consider the purpose and accuracy of measurements
Geodetic Parameters:	Refer to plate(s) titled 'Geodetic Parameters'
Vertical Datum:	■ For water depth: Lowest Astronomical Tide ■ For geotechnical investigation data: depth below seafloor

Data Acquisition System(s)

Primary System for Positioning Survey:	■ Starfix.G4+ solution generated from StarPack Port Receiver ■ GNSS positioning of vessel antennae ■ Subsurface positioning by ultra short baseline (USBL) system on borehole seabed frame relative to vessel
Primary System for Water Depth Measurement:	Conductivity, temperature and depth probe
Primary System for Investigation Data Points below Vertical Datum:	Refer to report appendices on investigation data for measurement of depth below seafloor, e.g. by depth (distance) transducer, inclinometer etc.
Secondary System(s):	■ GNSS positioning of vessel antennae (2 independent systems) ■ Water depth measurements from drill string ■ Water depth measurements from USBL system ■ Water depth measurements from echosounder

Results

Data Processing:	Fugro in-house software
Data Format(s):	PDF for viewing and printing (this primary document)
Conversion of Coordinates Systems:	Refer to plate(s) titled 'Geodetic Parameters'
Correlation of Depth Measurements:	Not considered, overall inspection of multiple data sets
Depth Correction(s) for Sloping or Irregular Seafloor:	Not considered
Depth Correction(s) for Very Soft Seabed:	Refer to report documents(s) on investigation data, where applicable
Positioning Survey and Depth Measurement Data:	■ Refer to plate(s) titled 'Coordinates and Water Depth'

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- Refer to report section(s) on investigation data for elevation (depth) of data points below vertical datum
- Presented water depth measurements serve to establish logging, testing and/or sampling depths below seafloor only

References

- ISO. (2003). *Measurement management systems - requirements for measurement processes and measuring equipment* (ISO 10012:2003). International Organization for Standardization. https://www.iso.org/standard/26033.html.
- ISO. (2014). *Petroleum and natural gas industries – specific requirements for offshore structures – part 8: marine soil investigations* (ISO 19901-8:2014). International Organization for Standardization. https://www.iso.org/standard/61145.html.

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Appendix B

Geotechnical Log

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B. Geotechnical Logs

B.1 Practice for Geotechnical Log

Approach	
Purpose:	Refer to Main Text
General Procedure:	▪ Refer to documents titled 'Geotechnical Log' and 'Geotechnical Parameter Values' presented in appendix titled 'Descriptions of Methods and Practices' ▪ According to ISO 19901-8:2014
Integration Level for Geotechnical Log:	▪ Level 1 Bundled Information ▪ Refer to document titled 'Geotechnical Log' in appendix titled 'Descriptions of Methods and Practices'

Results	
Data Processing and Interpretation:	▪ Discovery GeoVisual® software ▪ GeODin® software ▪ Graphical scales selected to suit general presentation of data ▪ No display of data outside of chart limits, i.e. some values may not be shown
Data Format(s):	PDF for viewing and printing (this primary document)
Matching Test, Sample and Logging Data:	Applicable
Ground Description: (sampling and in situ testing)	▪ According to ISO 19901-8:2014 and based on BS 5930:2015+A1:2020, ISO 14688-1:2017 and ISO 14688-2:2017 ▪ Where applicable, supplementary details according to document titled 'Soil Description and Classification' presented in appendix titled 'Descriptions of Methods and Practices'
Unit Weight derived from In Situ Test:	Not applicable
Relative Density derived from In Situ Test:	If applicable: ▪ refer to document titled 'Geotechnical Parameter Values' presented in appendix titled 'Description of Methods and Practices' ▪ according to Jamiolkowski et al. (2003) for saturated coarse-grained, frictional soil behaviour ▪ based on earth pressure coefficient values K_0 = 0.5 and K_0 = 1.0 ▪ relative density presented where soil behaviour type index I_c <2.60 or I_{SBT} <2.60 where applicable ▪ no calculation of relative density for initial penetration to a depth equivalent to five times diameter of deployed cone penetrometer

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	▪ presented values represent results of correlation(s), i.e. not an expected range
Undrained Shear Strength derived from In Situ Test:	If applicable: ▪ refer to document titled 'Geotechnical Parameter Values' presented in appendix titled 'Description of Methods and Practices' ▪ applies to interpreted fine-grained, cohesive soil behaviour ▪ based on cone factor of N_{kt} = 15 and N_{kt}= 20 ▪ target reference strength is laboratory consolidated (an)isotropic undrained triaxial compression on undisturbed sample, recompressed to estimated in situ stress conditions ▪ undrained shear strength presented where soil behaviour type index I_c >2.05 or I_{SBT} >2.05 where applicable ▪ no calculation of undrained shear strength for initial penetration to a depth equivalent to five times diameter of deployed cone penetrometer ▪ presented values represent results of correlation(s), i.e. not an expected range
Other Parameters Derived from In Situ Test:	Not applicable
Laboratory Test Data:	Refer to report section titled 'Geotechnical Laboratory Test Results' presented in Appendix D, for details on test values presented on geotechnical log
Other Derived Parameters/ Data:	Not applicable
Coordinates and Water Depth:	Applicable to borehole location
Water Depth Reference:	As inferred from conductivity, temperature, and depth probe measurements at start of testing, reduced to lowest astronomical tide
Depth Reference Correction:	None applied, downhole in situ testing and sampling continuous from start depth

B.2 Practice for Geotechnical Borehole

Borehole Stage Control	
General Procedure:	▪ According to ISO 19901-8:2014 ▪ Refer to document titled 'Geotechnical Borehole' in appendix titled 'Descriptions of Methods and Practices'
Set-up Stage:	Location as directed by client

Depth Reference Level:	Seafloor, particularly: ■ Seafloor inferred from CPT starting at about 0.3 m above base of seabed frame ■ No evidence for extremely soft ground at seafloor ■ Depth accuracy assessment of 'Vessel Drilling – Adverse'; refer to document titled 'Positioning Survey and Depth Measurement' presented in appendix titled 'Descriptions of Methods and Practices'
Drilling Stage:	Open-hole rotary drilling
Recording of Drilling Parameters:	Not applicable
Borehole Geophysical Logging:	Not applicable
Sampling and In Situ Testing:	■ Downhole CPT and push sampling ■ Sequence of in situ testing was adjusted to suit operations and site-specific conditions ■ Refer to geotechnical log(s) for details
Borehole Water Level Monitoring:	Not applicable
Borehole Termination Stage:	Reaching target borehole depth
Test Site Restoration:	■ No borehole backfill ■ Local seabed disturbance ■ Possibility of local seafloor depression(s) and drill cuttings on seafloor

Drilling	
Working Platform:	Geotechnical drilling vessel 'Gargano'
Drilling Method:	Open-hole rotary
Type and Make of Drill Rig:	R50 drill rig, load capacity 50 tonnes, Wirth power swivel
Drill Support:	SEACLAM seafloor template for lateral support at seafloor during drilling and for clamping of drill pipe during logging, testing, and sampling
Drill Pipe:	API-type, 4.5 inch ID
Drill Bit:	Tungsten carbide casing shoe for open-hole drilling
Core Barrel:	Not applicable
Drill Fluid:	Water, guar gum, and Pure-Bore®
Initial Borehole Orientation:	Vertical

Cone Penetration Test
Refer to 'Practice for Cone Penetration Test' presented in appendix titled 'Results of Cone Penetration Test'

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Push Sampling	
Sampling System:	Downhole WIPSAMPLER®
Sampler Insertion Equipment:	Jacking unit with maximum thrust capacity of up to 80 kN and penetration rate of about 20 mm/s
Reaction Equipment:	Self-weight of drill pipes, drill collars and SEACLAM seabed frame
Open-Tube Sampler:	▪ Thick-walled 3 inch flush cylindrical sample tube, 80 mm OD, 72 mm ID ▪ Thin-walled 3 inch flush cylindrical sample tube, 76 mm OD, 72 mm ID ▪ Thick-walled 2 inch cylindrical sample tube, 60 mm OD, 53 mm ID
Core Catcher:	Applicable in case of sample recovery problems
Push Sampling Termination:	Whichever occurs first: ▪ reaching maximum permissible sample tube penetration ▪ reaching maximum capacity of sample insertion equipment and/or sample tube ▪ reaching maximum capacity of reaction equipment ▪ circumstances at the discretion of the equipment operator, such as risk of equipment damage or risk to personnel safety

Sample Handling
Refer to 'Practice for Sample Handling' presented in appendix titled 'Geotechnical Laboratory Test Results'

B.3 Comments on Results

This section presents a geotechnical log based on results of downhole cone penetration tests (CPTs) supported by sample information where applicable.

Strata descriptions presented in the geotechnical log are an interpretation of available data sources, including sample descriptions, sample photographs, laboratory tests, and in situ tests. The strata descriptions may differ from the individual sample descriptions presented on 'Laboratory Classification Test Results' plates in appendix titled 'Geotechnical Laboratory Test Results'. The sample descriptions are based on onsite visual classification of individual samples, whereas strata descriptions consider additional data sources.

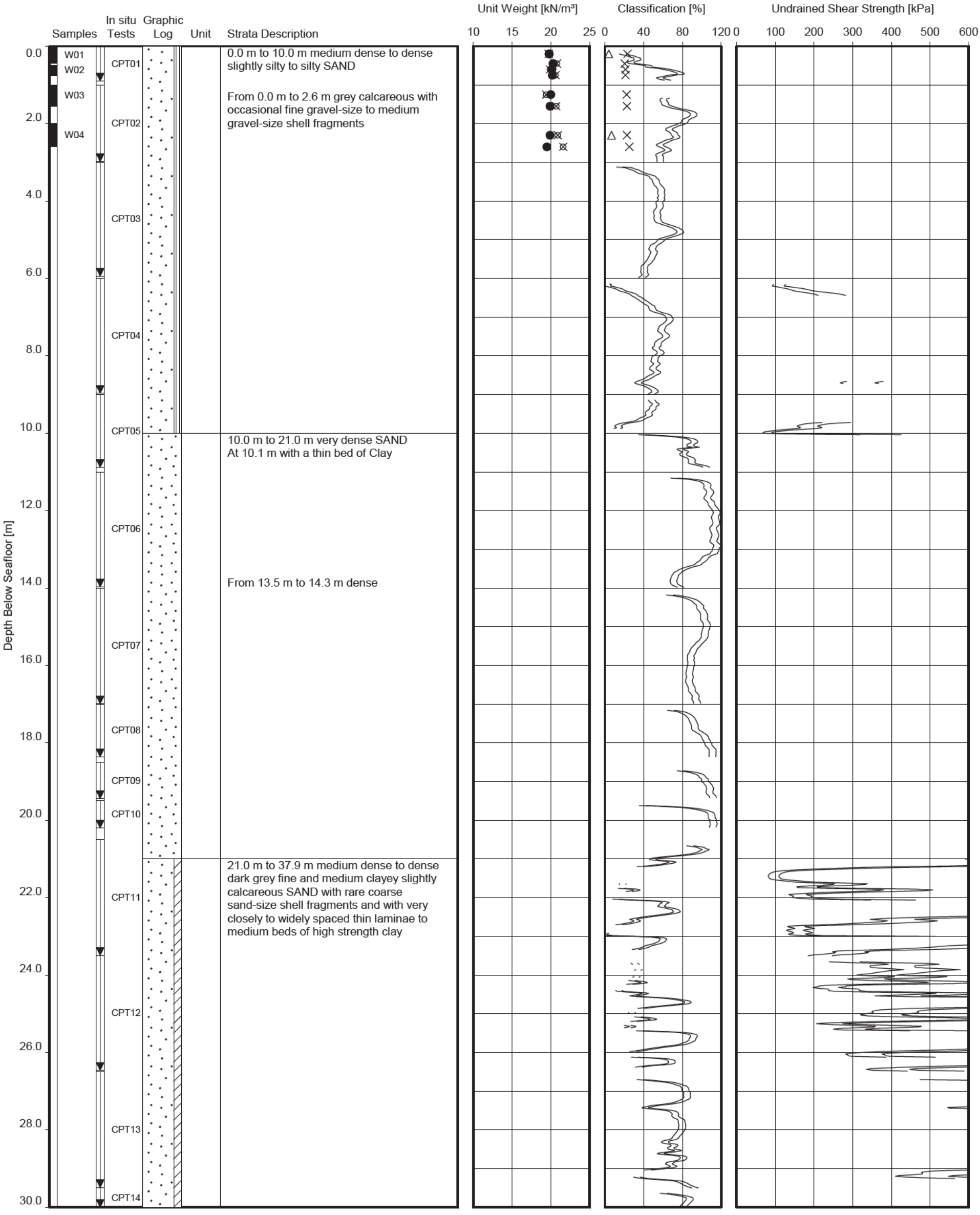
The graphic log based on strata descriptions in the geotechnical log represents the primary and secondary soil fractions for layers present at a particular location. It should be noted that the graphic representation of strata containing frequent thin laminae to thin beds is a simplification of the complex nature of layers containing such features. Additionally, strata

with a thickness < 0.6 m are not presented as separate main soil layers nor as sub-layers in the graphic log, unless the strata are considered relevant for engineering purposes.

B.4 References

- BSI. (2020). *Code of practice for ground investigations* (BS 5930:2015+A1:2020). British Standards Institution. https://shop.bsigroup.com/products/code-of-practice-for-ground-investigations/standard
- Computer Program Discovery GeoVisual. *Recording, presentation, and analysis of in situ geodata*.
- ISO. (2014). *Petroleum and natural gas industries - Specific requirements for offshore structures - Part 8: marine soil investigations* (ISO 19901-8:2014). International Organization for Standardization. https://www.iso.org/standard/61145.html
- ISO. (2017). *Geotechnical investigation and testing – identification and classification of soil – part 1: identification and description* (ISO 14688-1:2017). International Organization for Standardization. https://www.iso.org/standard/66345.html
- ISO. (2017). *Geotechnical investigation and testing – identification and classification of soil – part 2: principles for a classification* (ISO 14688-2:2017). International Organization for Standardization. https://www.iso.org/standard/66346.html
- Jamiolkowski, M., Lo Presti, D.C.F. & Manassero, M. (2003). Evaluation of relative density and shear strength of sands from CPT and DMT. In J.T. Germaine, T.C. Sheahan & R.V. Whitman (Eds.). *Soil behavior and soft ground construction* (pp. 201–238). American Society for Civil Engineers. https://doi.org/10.1061/9780784406595

GeODin/Composite Geotechnical Log (A3 Portrait) 0-30.0m.GLO/2023-08-02 18:10



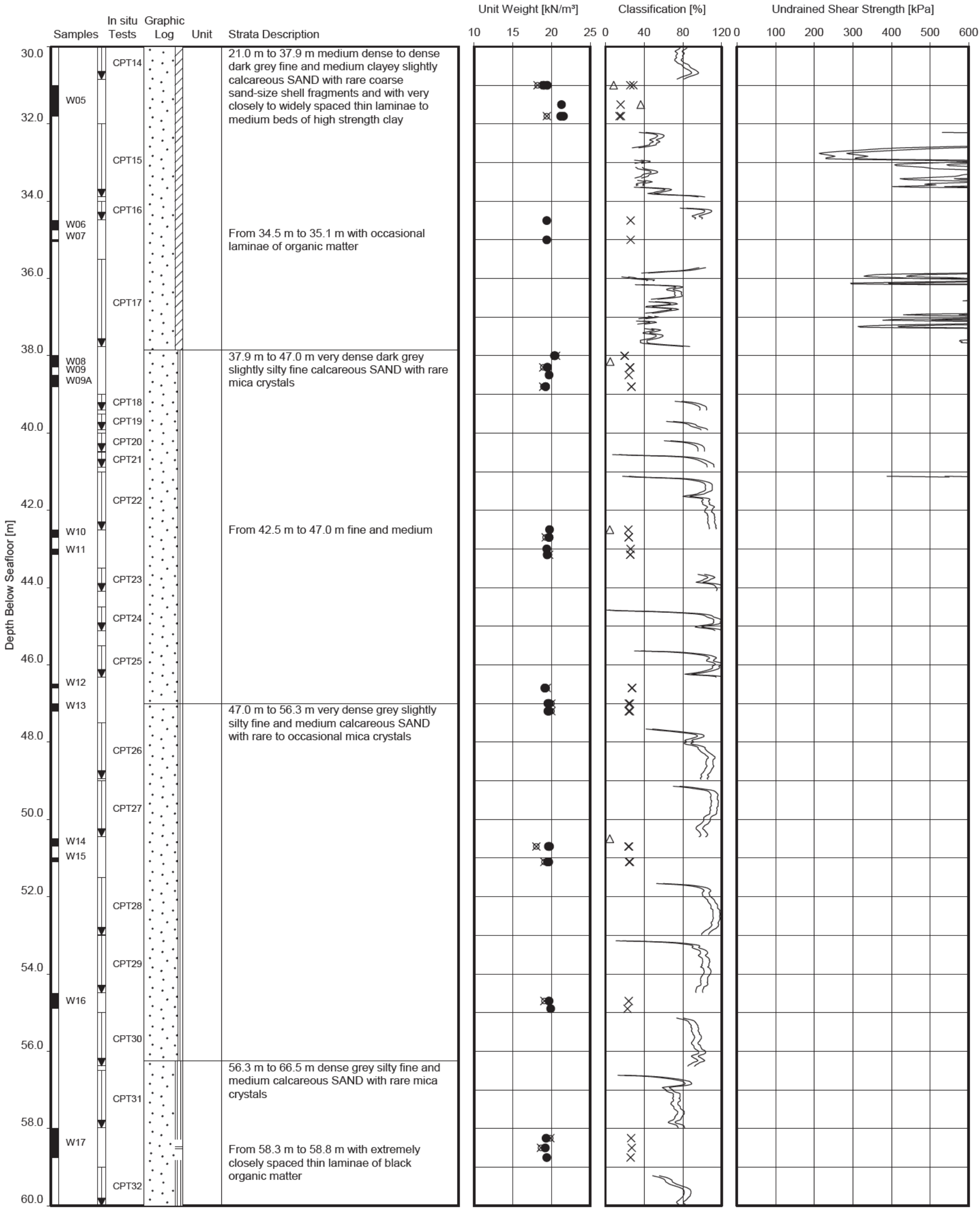
Date commenced : 26-Feb-2023
Method : rotary borehole drilling, sampling and testing
Recovery depth : to 100.7 m below seafloor
Penetration depth : to 100.7 m below seafloor
Water depth : 25.4 m
Coordinates : 586494 m E 5931306 m N

Location(s):
NEPL7-BH01

- Derived from water content
- Derived from volume mass calculation
- Water content
- Plastic limit
- Liquid limit
- Plasticity index
- Percentage fines
- Carbonate content
- Organic content
- Relative density derived from CPT
- Pocket penetrometer
- Torvane
- Fallcone
- Hand vane
- Laboratory vane
- UU-triaxial
- CU-triaxial (q_{max})
- CU-triaxial (q/p'_{max})
- Direct simple shear
- In situ vane shear test
- Undrained shear strength derived from CPT
- Slashed symbol refers to test on remoulded soil

GEOTECHNICAL LOG

GeODin/Composite Geotechnical Log (A3 Portrait) 0-30.0m.GLO/2023-08-02 18:10

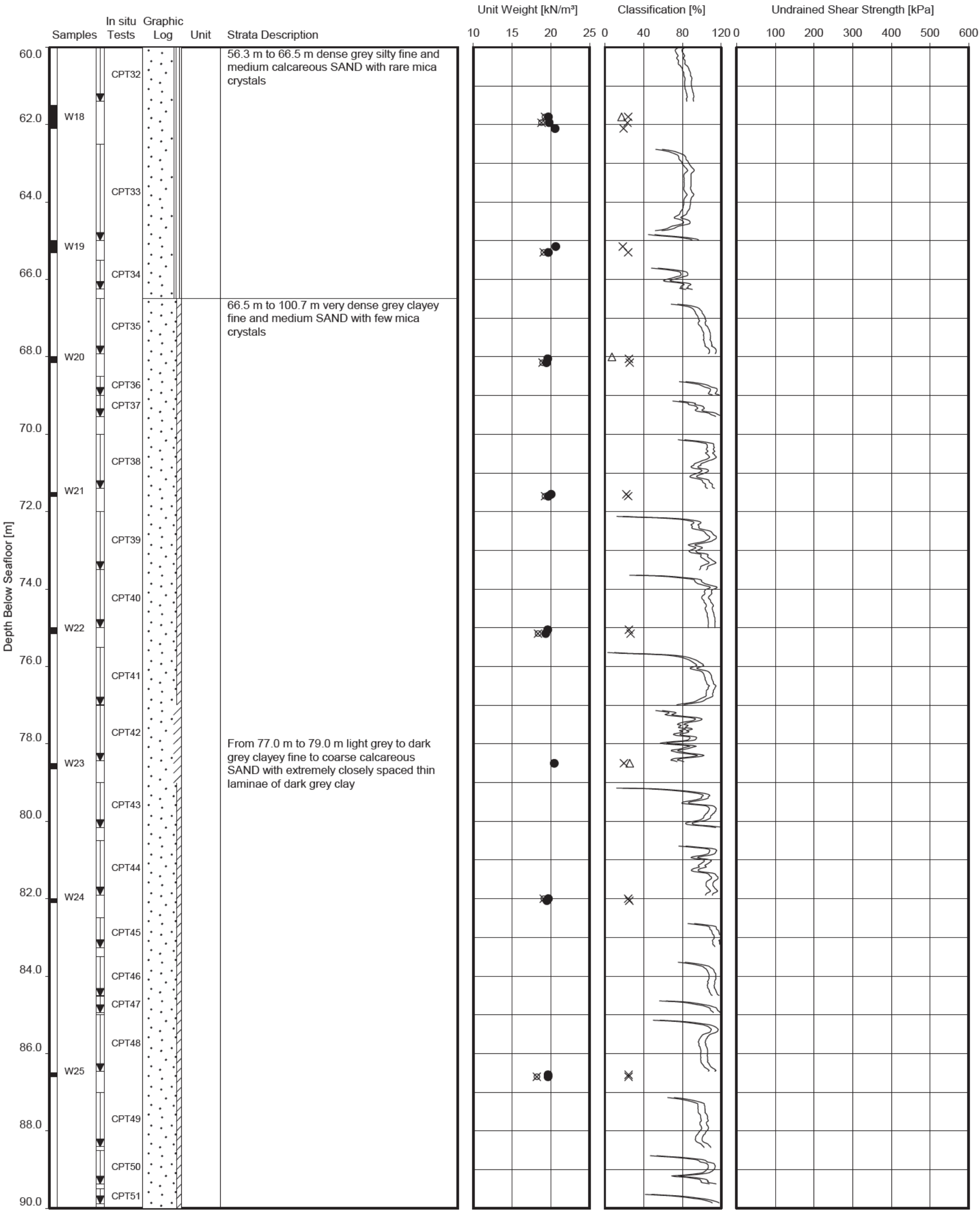


Date commenced : 26-Feb-2023
Method : rotary borehole drilling, sampling and testing
Recovery depth : to 100.7 m below seafloor
Penetration depth : to 100.7 m below seafloor
Water depth : 25.4 m
Coordinates : 586494 m E 5931306 m N

Location(s):
NEPL7-BH01

- Derived from water content
- Derived from volume mass calculation
- Water content
- Plastic limit
- Liquid limit
- Plasticity index
- Percentage fines
- Carbonate content
- Organic content
- Relative density derived from CPT
- Pocket penetrometer
- Torvane
- Fallcone
- Hand vane
- Laboratory vane
- UU-triaxial
- CU-triaxial (q_{max})
- CU-triaxial (q/p'_{max})
- Direct simple shear
- In situ vane shear test
- Undrained shear strength derived from CPT
- Slashed symbol refers to test on remoulded soil

GEOTECHNICAL LOG



Date commenced : 26-Feb-2023
Method : rotary borehole drilling, sampling and testing
Recovery depth : to 100.7 m below seafloor
Penetration depth : to 100.7 m below seafloor
Water depth : 25.4 m
Coordinates : 586494 m E 5931306 m N

Location(s):
NEPL7-BH01

- Derived from water content
- Derived from volume mass calculation
- Water content
- Plastic limit
- Liquid limit
- Plasticity index
- Percentage fines
- Carbonate content
- Organic content
- Relative density derived from CPT
- Pocket penetrometer
- Torvane
- Fallcone
- Hand vane
- Laboratory vane
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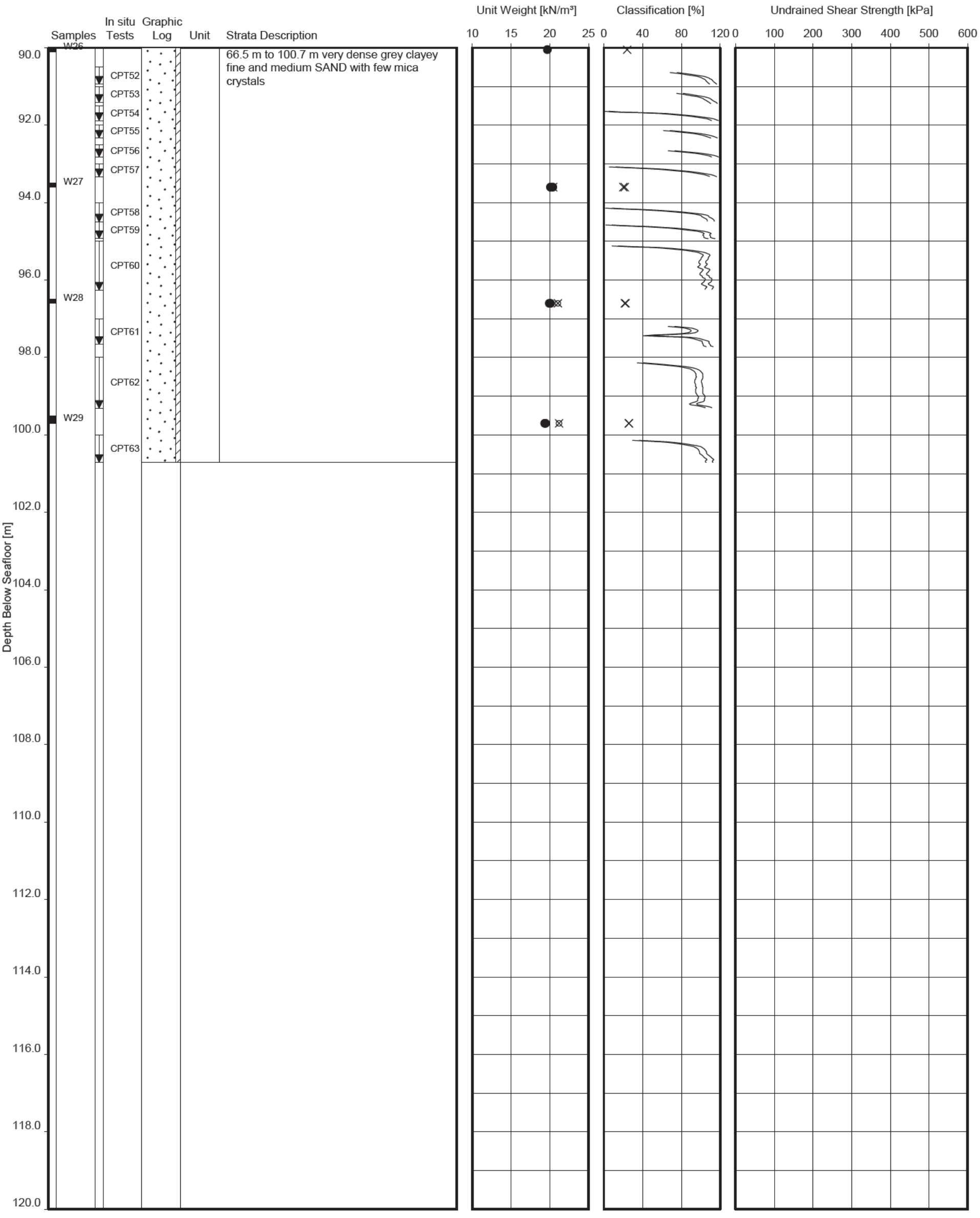
GEOTECHNICAL LOG

GeODin/Composite Geotechnical Log (A3 Portrait) 0-30.0m.GLO/2023-08-02 18:10



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GEOTECHNICAL LOG

Appendix C

Results of Cone Penetration Tests

Contents Appendix C: Results of Cone Penetration Tests

Page

C. Results of Cone Penetration Tests 1
C.1 Practice for Cone Penetration Test 1
C.2 Comments on Results 2
C.3 References 3

List of Plates

Title	Plate
Cone Penetration Test Results	C-1 to C-38
Cone Penetration Test; q_c, f_s, u_2	
Cone Penetration Test; q_t, q_n, R_f, B_q	
Cone Penetration Test; I_c, I_{SBT}, Q_{tn}, F_r, γ	
Cone Penetration Test; Zero Drift	

C. Results of Cone Penetration Tests

C.1 Practice for Cone Penetration Test

Test Overview	
General Procedure:	▪ According to ISO 19901-8:2014 for test practice ▪ According to ISO 10012:2003 for metrological confirmation ▪ Refer to document titled 'Cone Penetration Test' presented in appendix titled 'Descriptions of Methods and Practices'
Target Application Class:	Class 3 of ISO 19901-8:2014, refer to document titled 'Cone Penetration Test' presented in the appendix titled 'Descriptions of Methods and Practices'
Set-up Stage:	▪ Location as directed by client ▪ Cone penetrometer selected as directed by client and in liaison with Fugro
Additional Measurements:	Not applicable
Test Stage:	Refer to document titled 'Geotechnical Borehole' presented in the appendix titled 'Descriptions of Methods and Practices'
Test Termination:	Refer to document titled 'Cone Penetration Test' presented in the appendix titled 'Descriptions of Methods and Practices'
Drill-Out:	Not applicable
Test Site Restoration:	▪ No backfill of test hole ▪ Local seabed disturbance ▪ Possibility of local seafloor depression

CPT Apparatus	
Thrust Machine:	WISON® Mk V wireline hydraulic jacking unit, nominal 100 kN thrust capacity and 1.5 m (CP5) or 3 m (CP10) stroke
Reaction Equipment:	Self-weight of drill pipes, drill collars and SEACLAM seabed frame
Push Rod:	25.5 mm (CP5) and 36 mm (CP10) push rod outer diameter
Penetrometer Type:	▪ Type CP10-CF80PB20-P1E1M1-V1 piezocone penetrometer, 80 kN load sensors (100 kN for overloading), 20 MPa pressure sensor, HDPE filter in cylindrical extension above base of cone, 1000 mm^2 cone base area, 15 000 mm^2 sleeve area ▪ Type CP5-CF50-P1E1M2-V1 friction cone penetrometer, 50 kN load sensors (80 kN for overloading), 500 mm^2 cone base area, 7500 mm^2 sleeve area ▪ Net area ratio for cone tip as per plate(s) titled 'Cone Penetration Test – Zero Drift'

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Test Results	
Data Processing and Management:	▪ Refer to document titled 'Cone Penetration Test' presented in appendix titled 'Descriptions of Methods and Practices' ▪ Discovery GeoVisual® software ▪ Graphical scales selected to suit general presentation of data and requirements of standards, where practicable ▪ No display of data outside of chart limits, i.e. some values may not be shown
Data Format(s):	PDF for viewing and printing (this primary document)
Water Depth Reference:	As inferred from conductivity, temperature, and depth measurements at start of testing, reduced to lowest astronomical tide
Depth Reference Level:	Seafloor, refer to sub-section 'Practice for Geotechnical Borehole' in appendix titled 'Geotechnical Logs' for depth accuracy assessment
Depth Correction for Penetrometer Inclination:	Not applicable
Parameter Values for Data Processing:	▪ Refer to plate(s) titled 'Cone Penetration Test Results' showing soil behaviour type index, soil unit weight and supplementary normalised parameter values ▪ Constant value of soil unit weight of 20 kN/m^3 ▪ Hydrostatic pore pressure conditions with zero at seafloor and unit weight of (pore)water of 10 kN/m^3

C.2 Comments on Results

Typically, CPTs show negative pore pressures upon penetration of dense to very dense and/or (silty) fine sands. This is not uncommon. Loss of saturation usually causes a negative or sluggish pore pressure response during penetration of ground below the zone causing desaturation of the pore pressure filter around the pore pressure sensor. Negative pore pressure measurements affect derived values such as net cone resistance and pore pressure ratio. The influence of negative pore pressure is to be considered when using the derived values. Refer to the document titled 'Geotechnical Parameter Values' in the appendix titled 'Descriptions of Methods and Practices' for details.

Deck measurements, referred to as 'zero reading', were taken before the start and after the end of each test to verify the performance of the cone penetrometer during testing. Comparison of zero load measurements is used to assess the cone penetrometer and equipment performance. Plates titled 'Cone Penetration Test - Zero Drift' present the results of this comparison.

At CPT14 at 29.5 m below seafloor (BSF), no full penetration could be achieved because latching was temporarily lost after starting the test. Data for the affected interval is not presented.

After performing CPT56 at 92.5 m BSF, communication was lost with the cone penetrometer and therefore zero drift could not be determined.

C.3 References

- Computer Program Discovery GeoVisual. *Processing, presentation, and analysis of in situ test data.*
- ISO. (2003). *Measurement management systems - Requirements for measurement processes and measuring equipment* (ISO 10012:2003). International Organization for Standardization. https://www.iso.org/standard/26033.html
- ISO. (2014). *Petroleum and natural gas industries - Specific requirements for offshore structures - Part 8: Marine soil investigations* (ISO 19901-8:2014). International Organization for Standardization. https://www.iso.org/standard/61145.html

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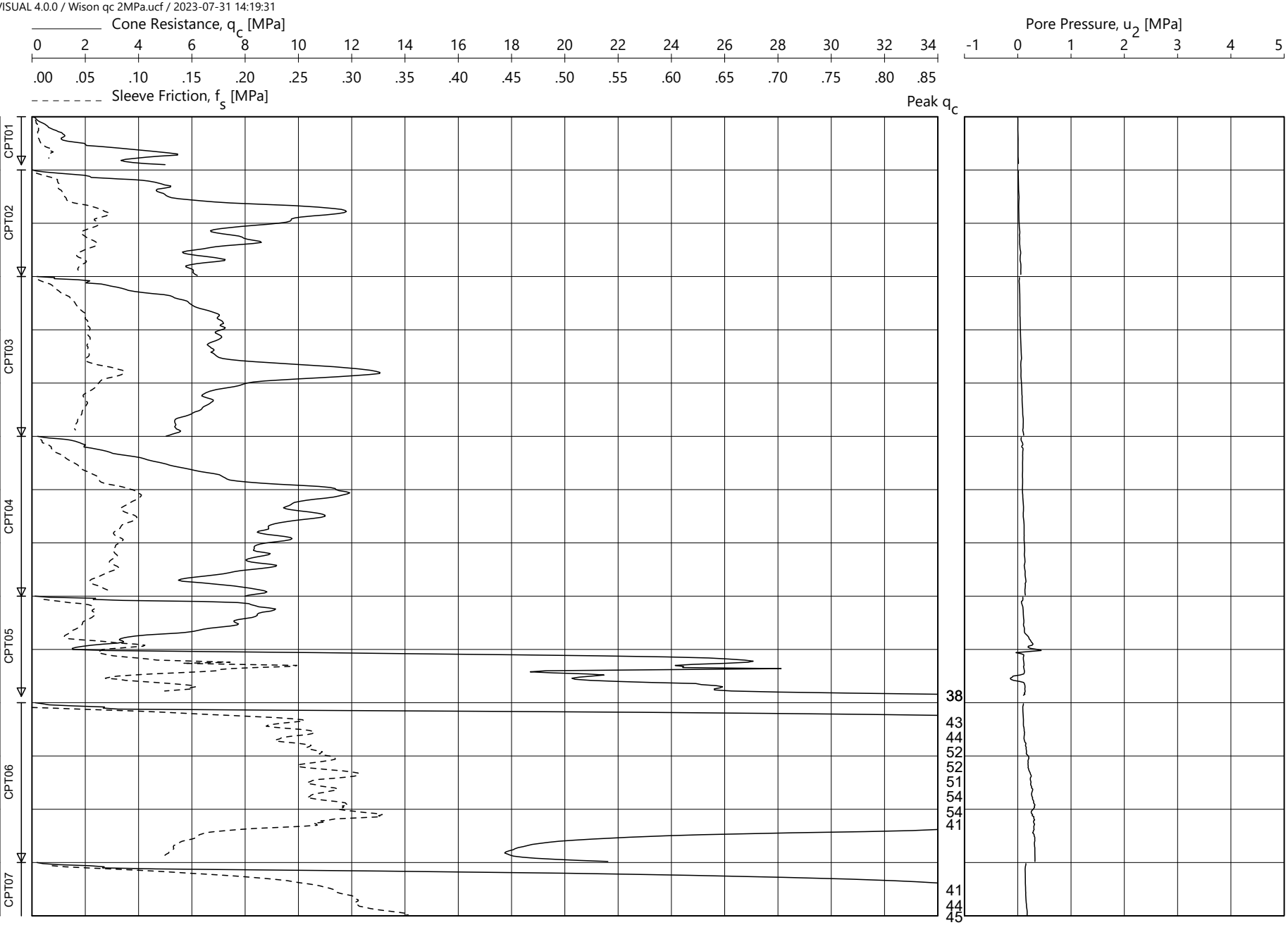

Date Of Testing : 26-Feb-2023
Water Depth [m LAT] : 25.4
Coordinates [m] : E 586494 N 5931306

CONE PENETRATION TEST
NEPL7-BH01

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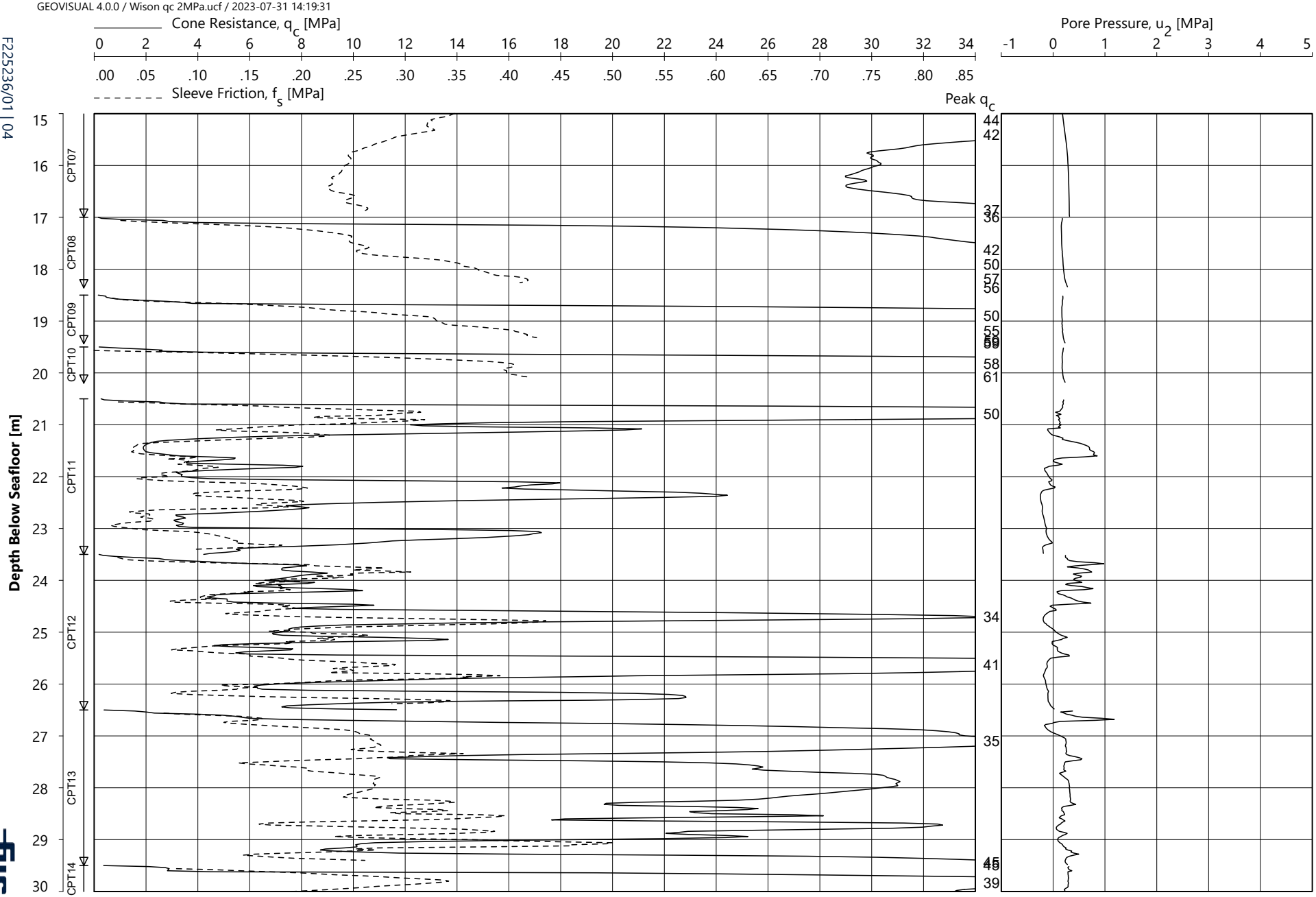

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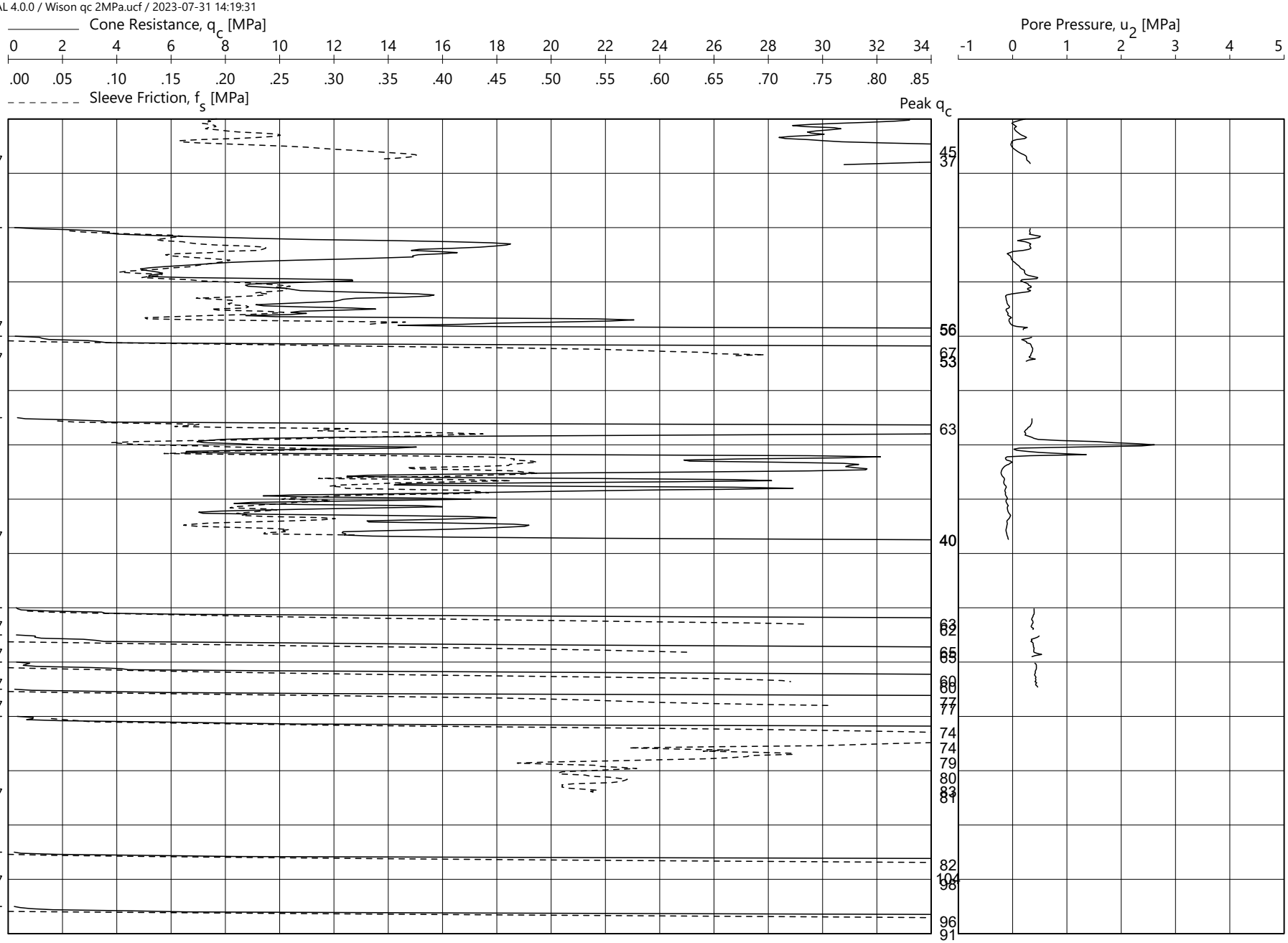

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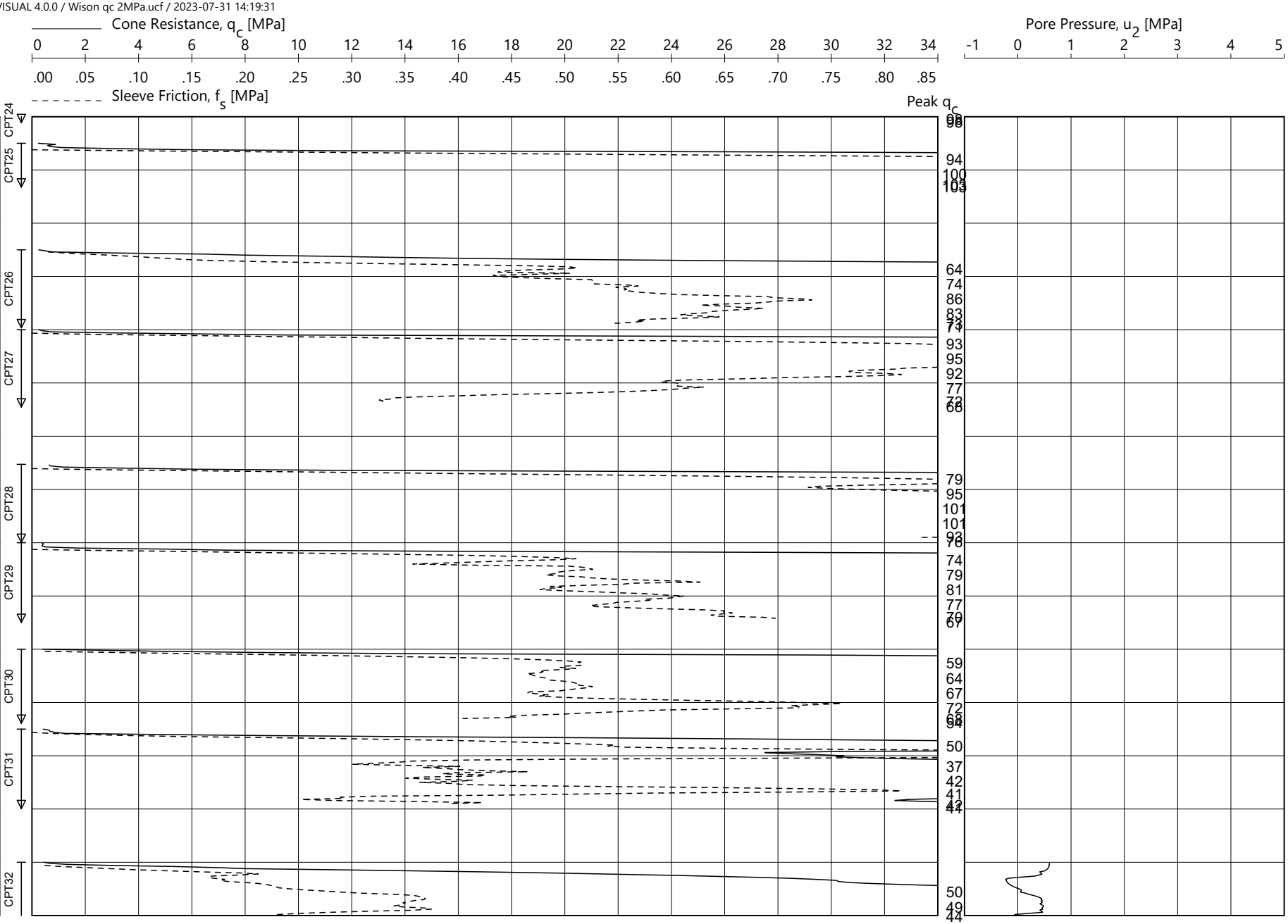

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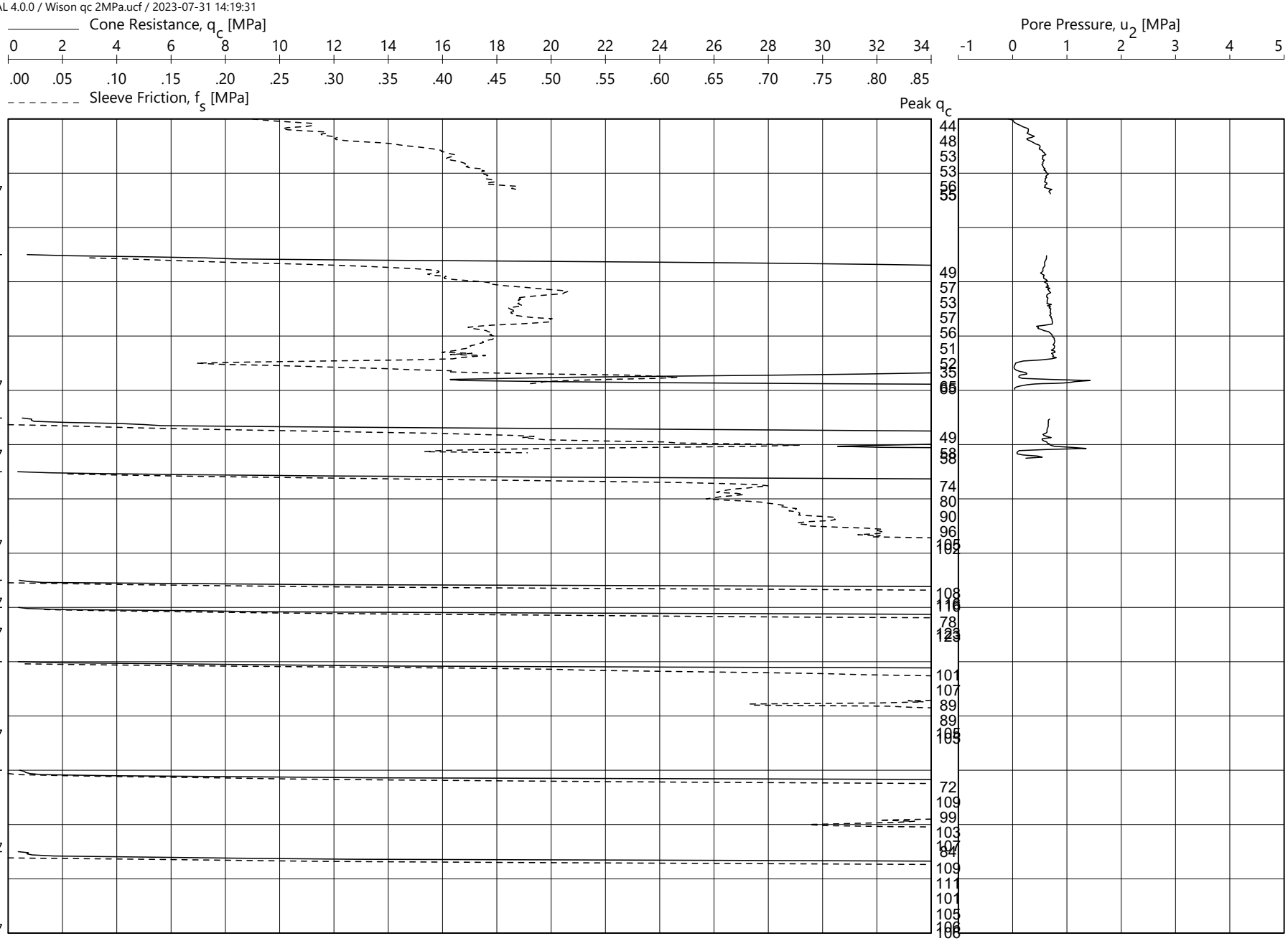

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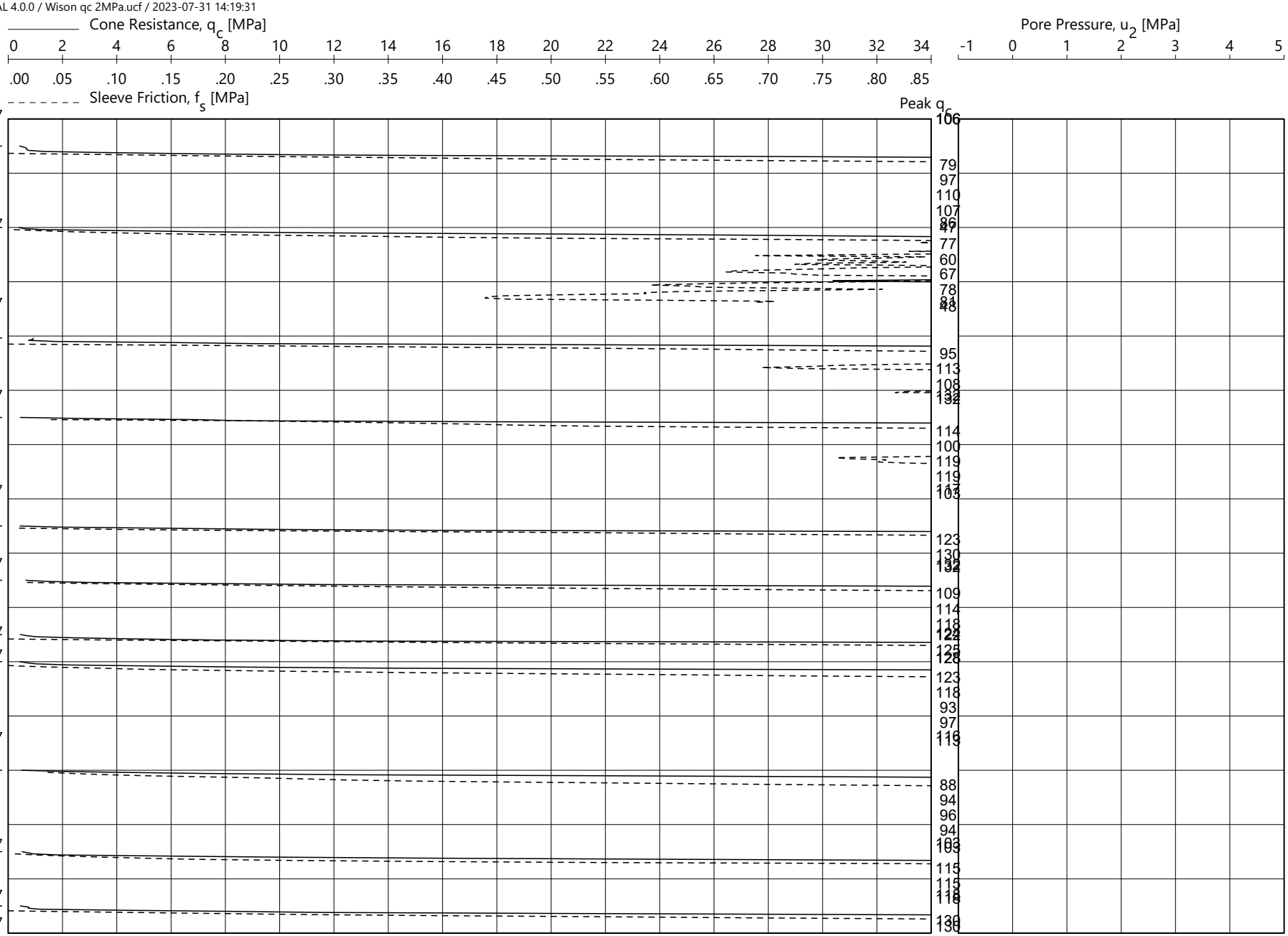

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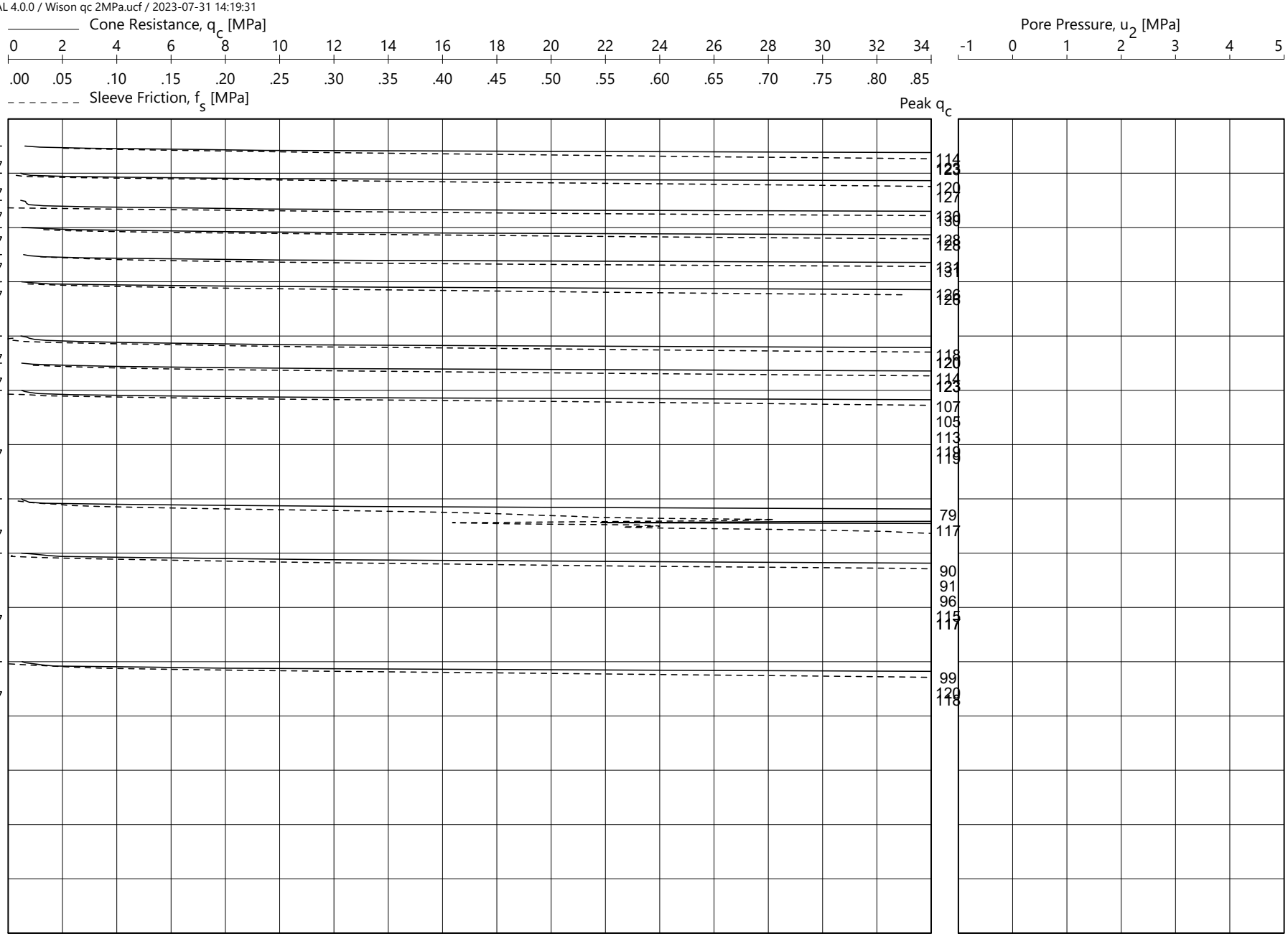

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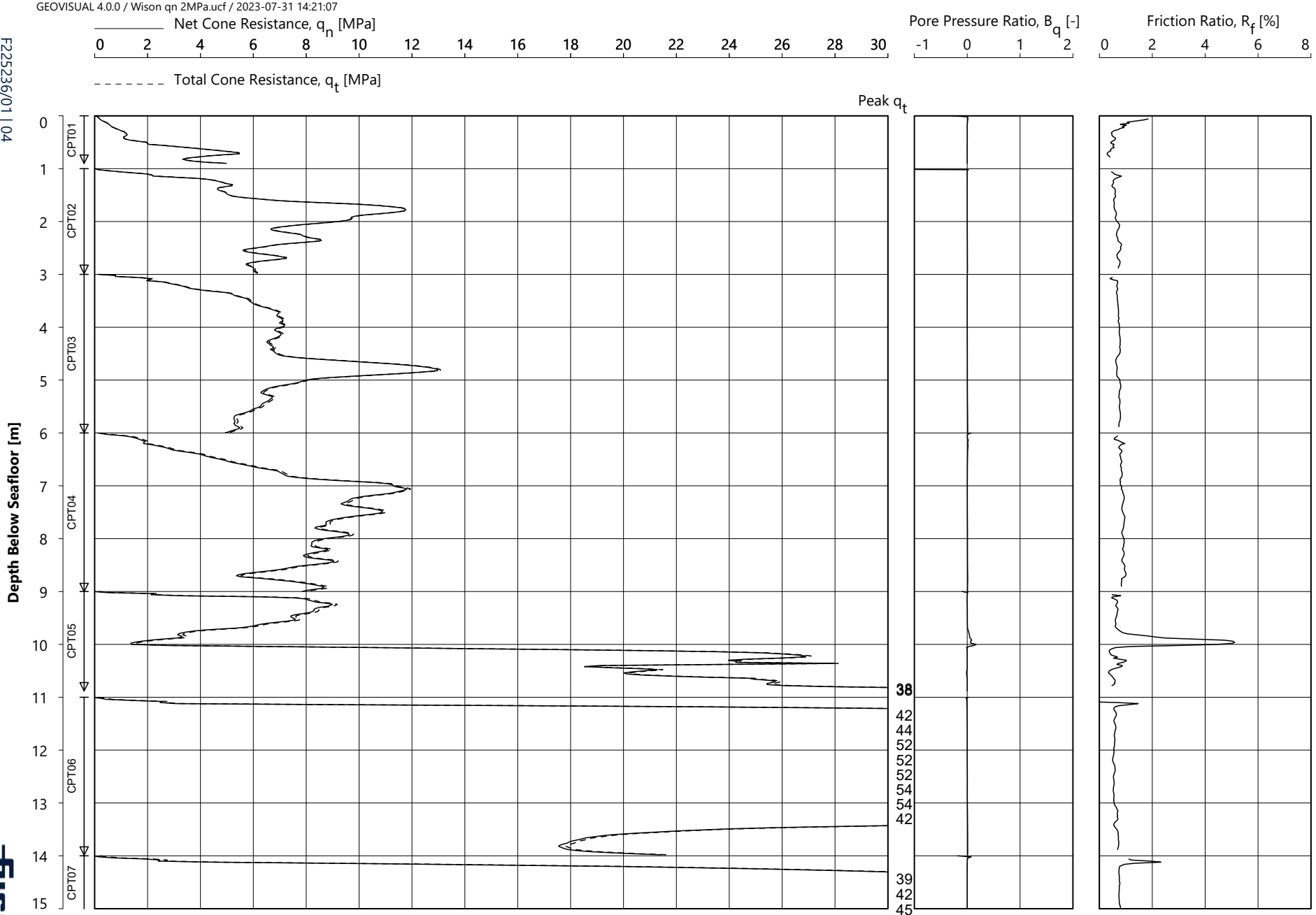

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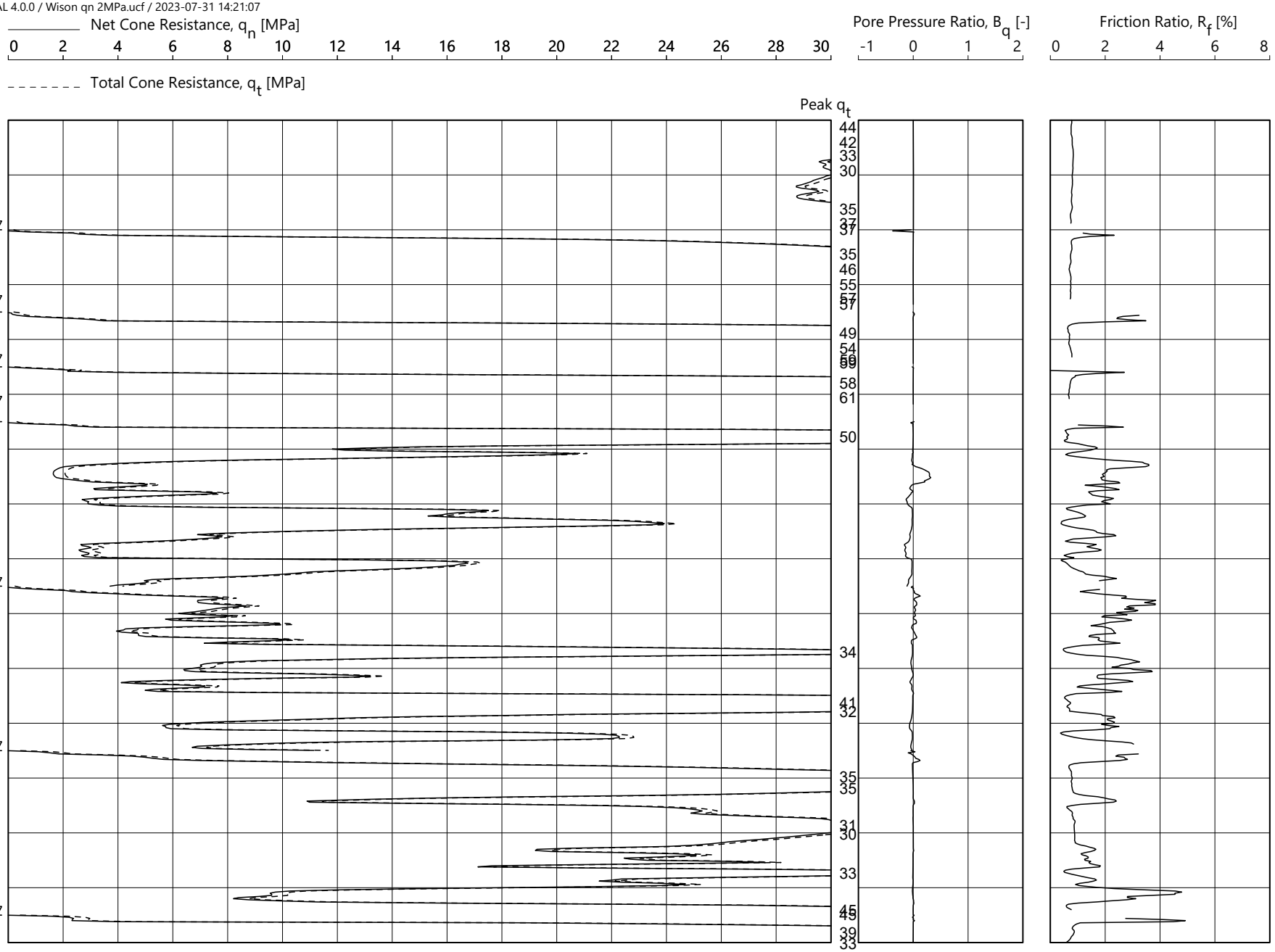

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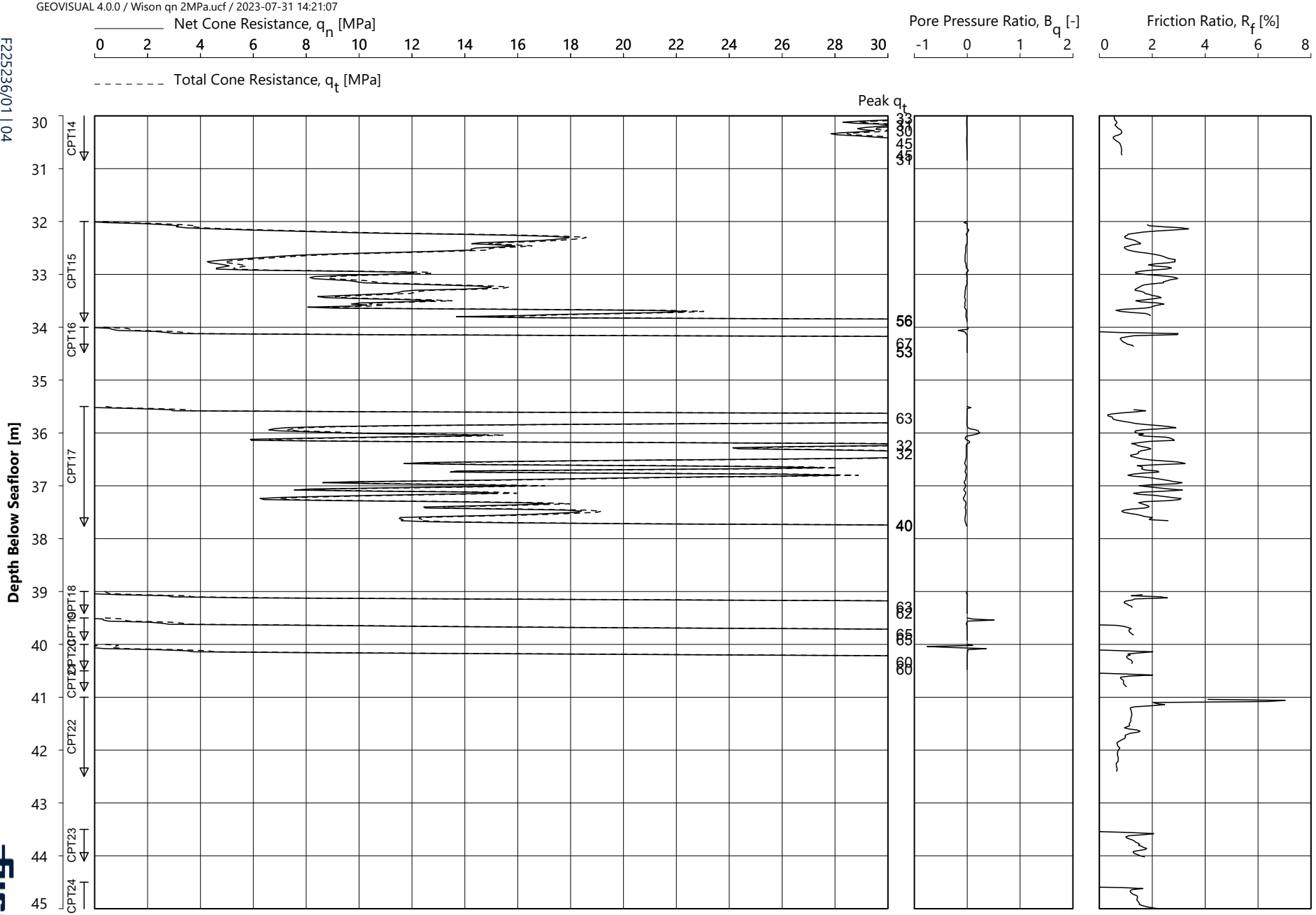

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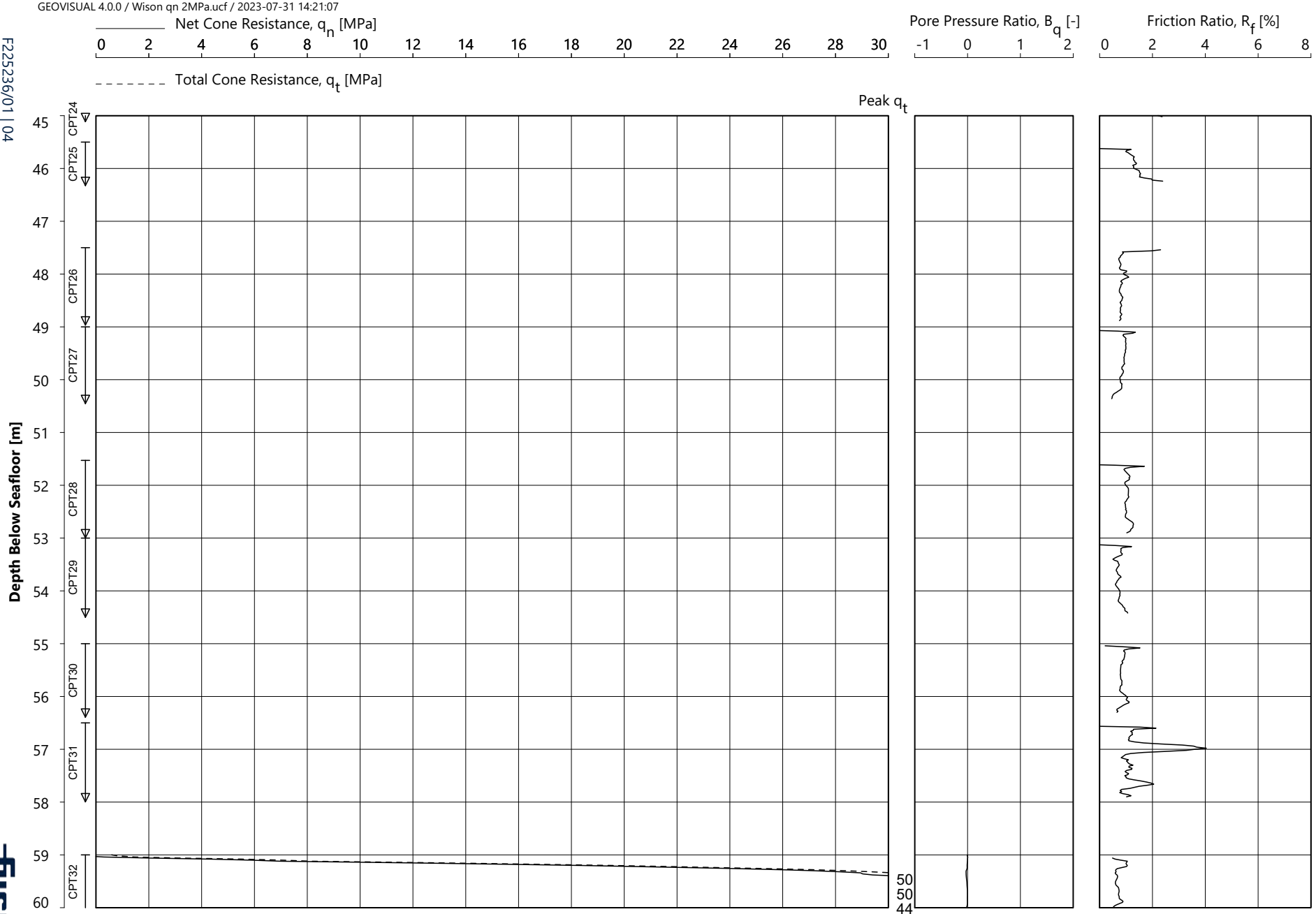

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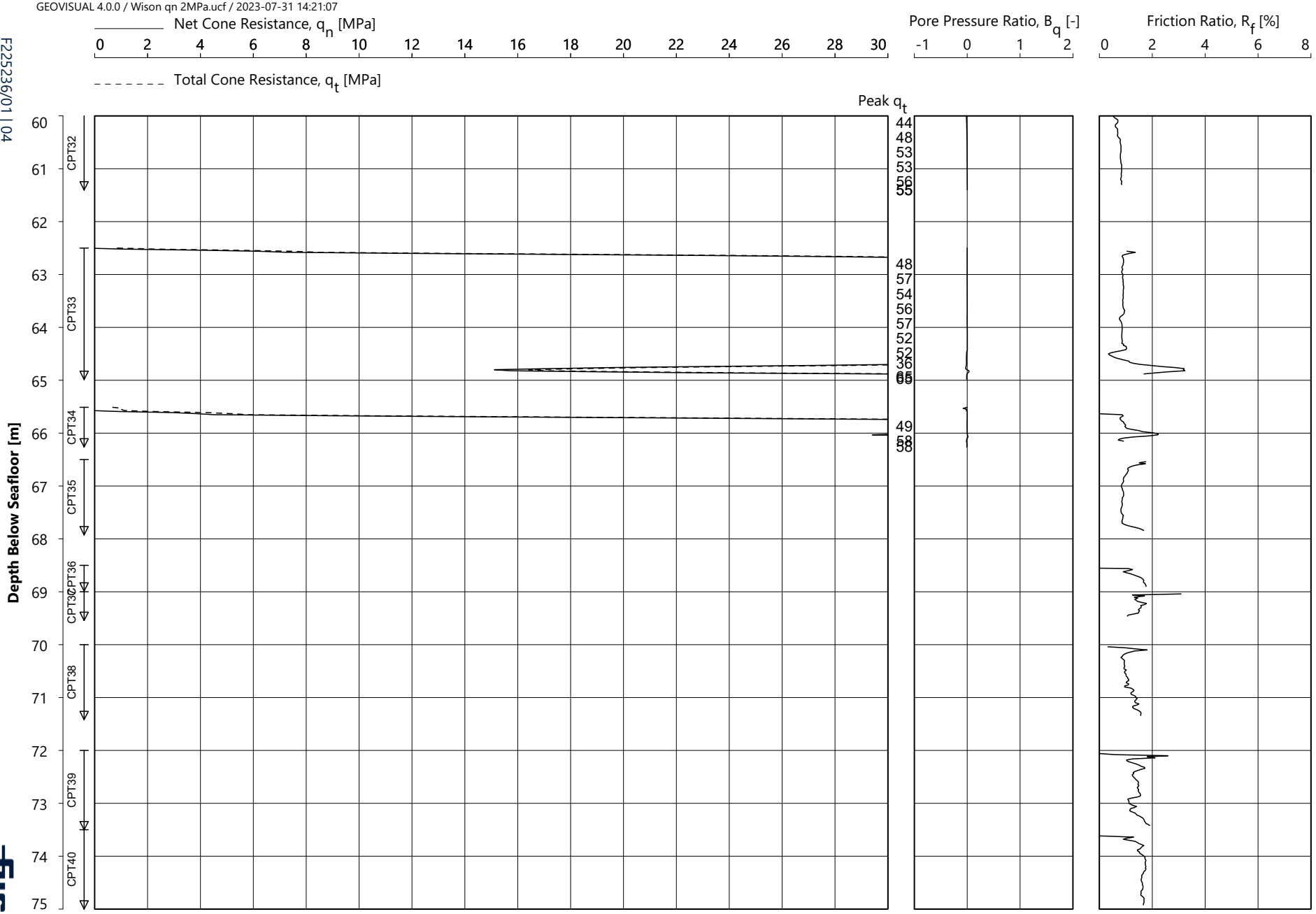

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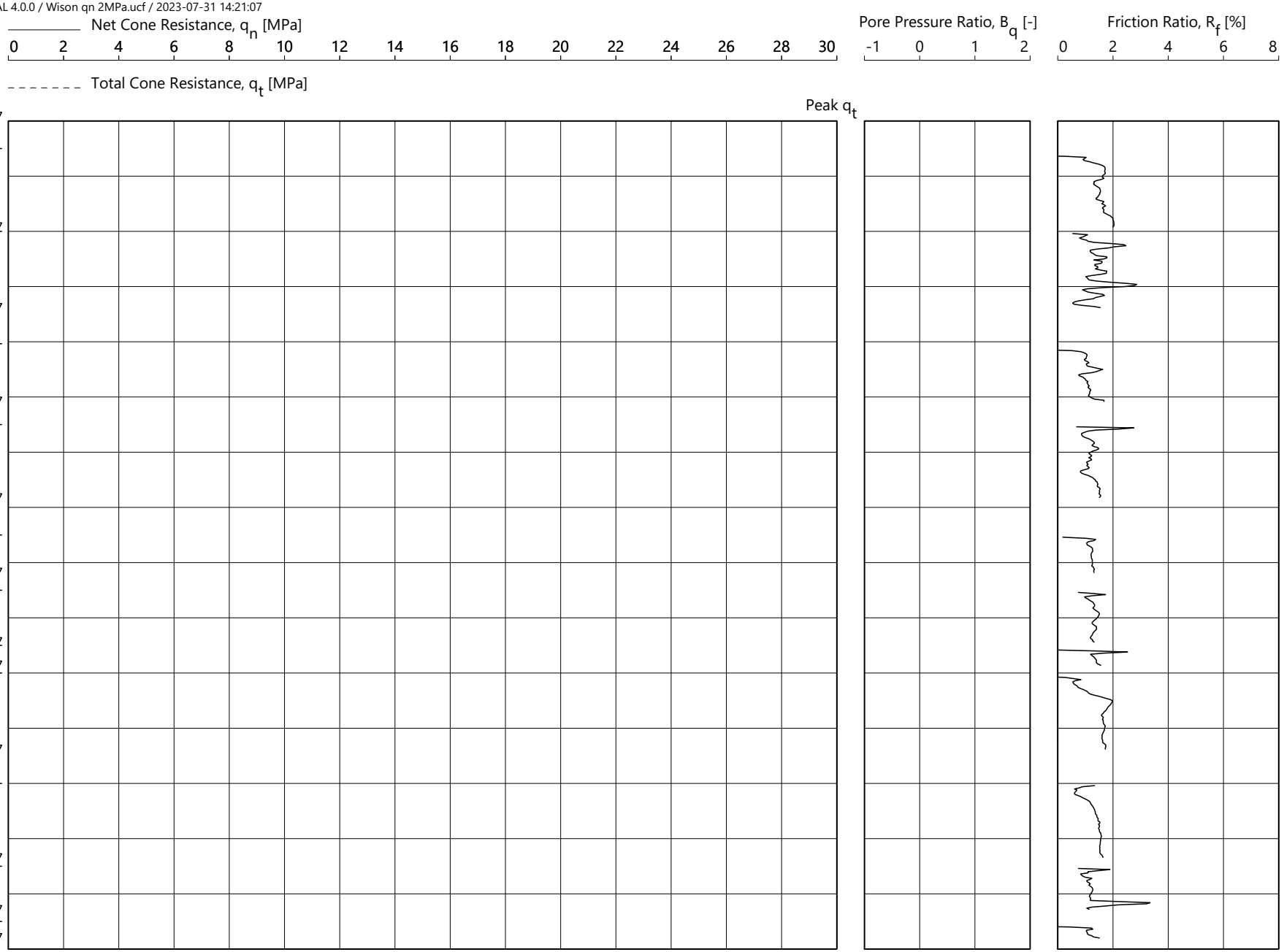

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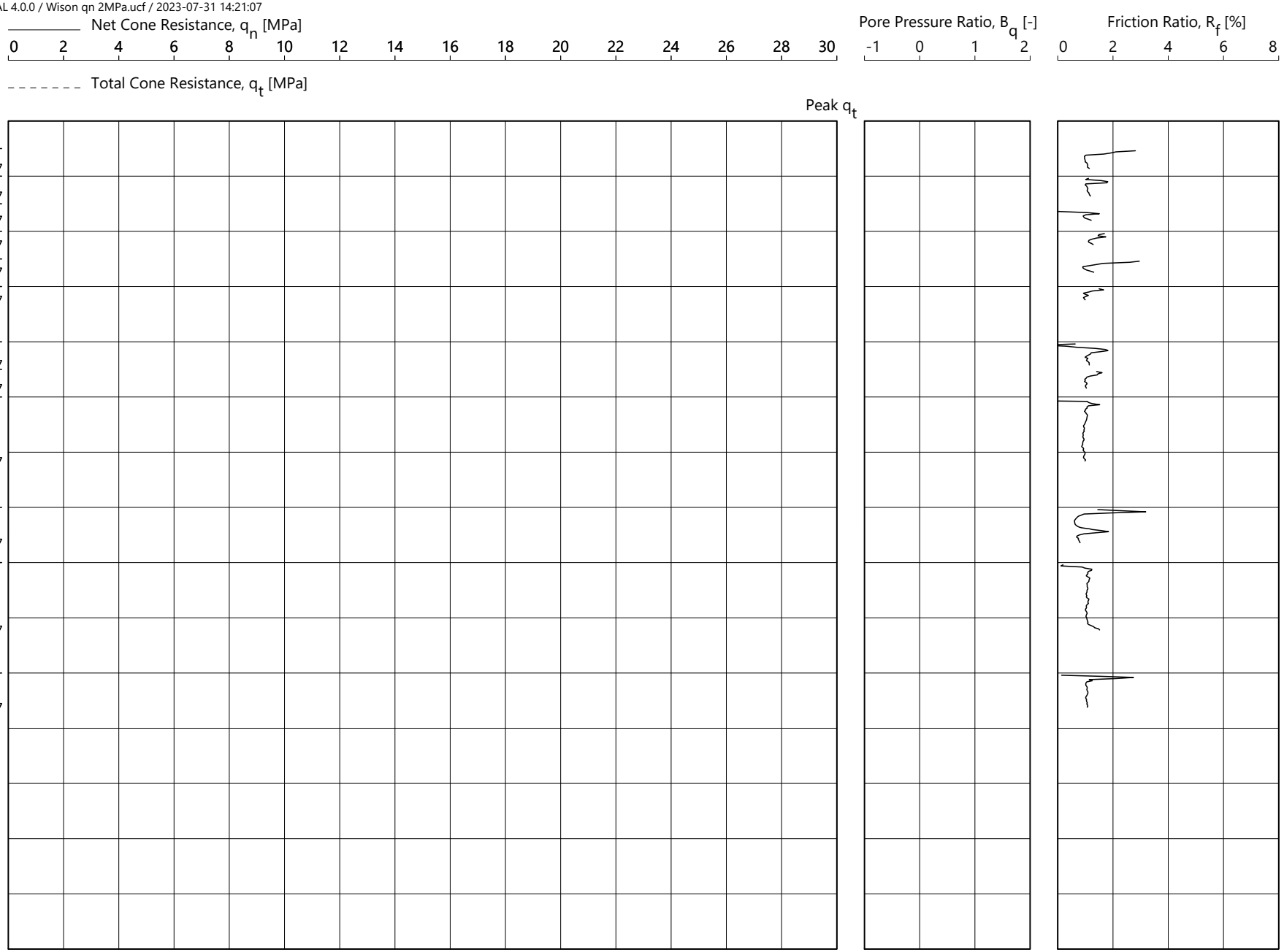

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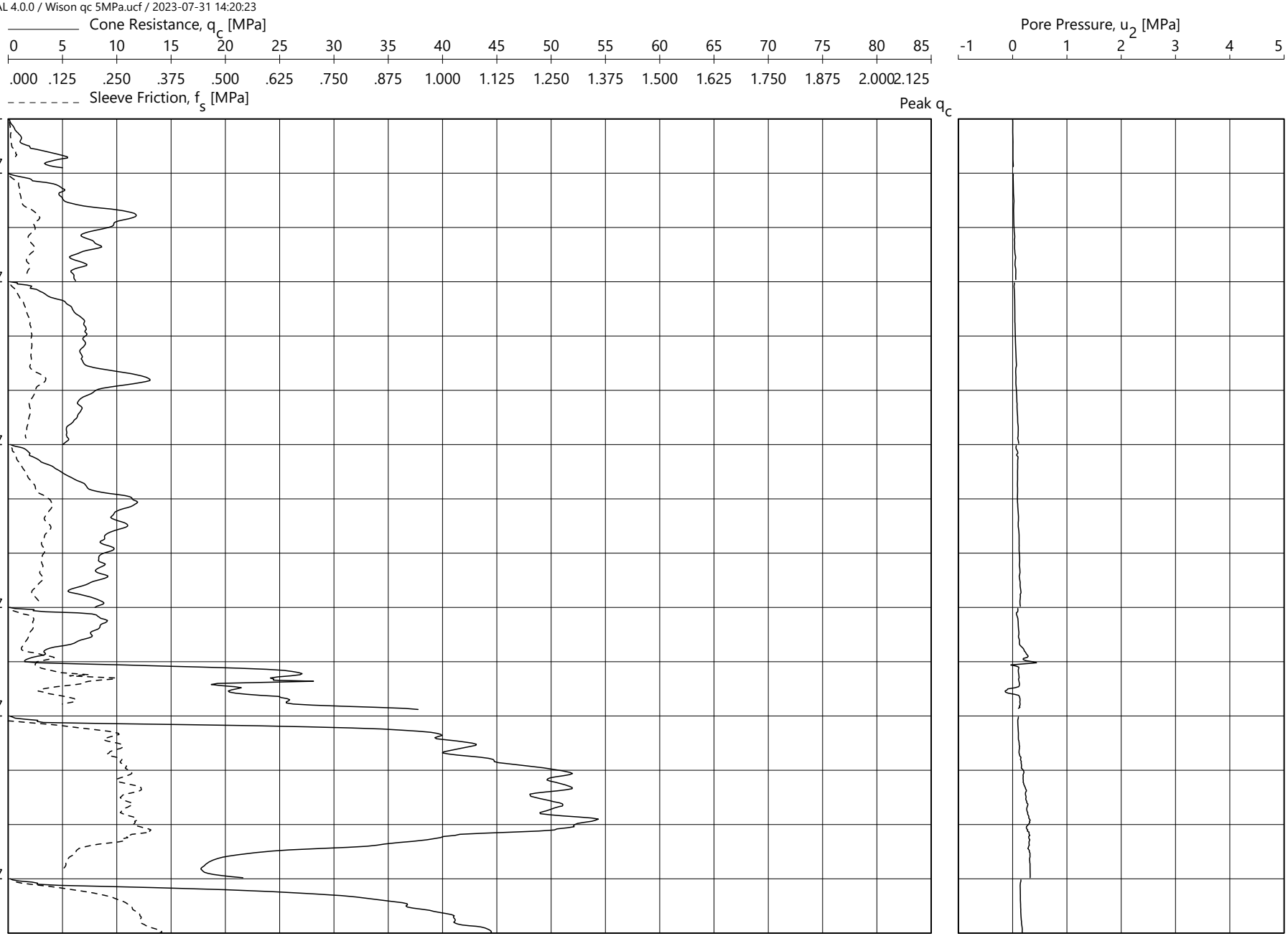

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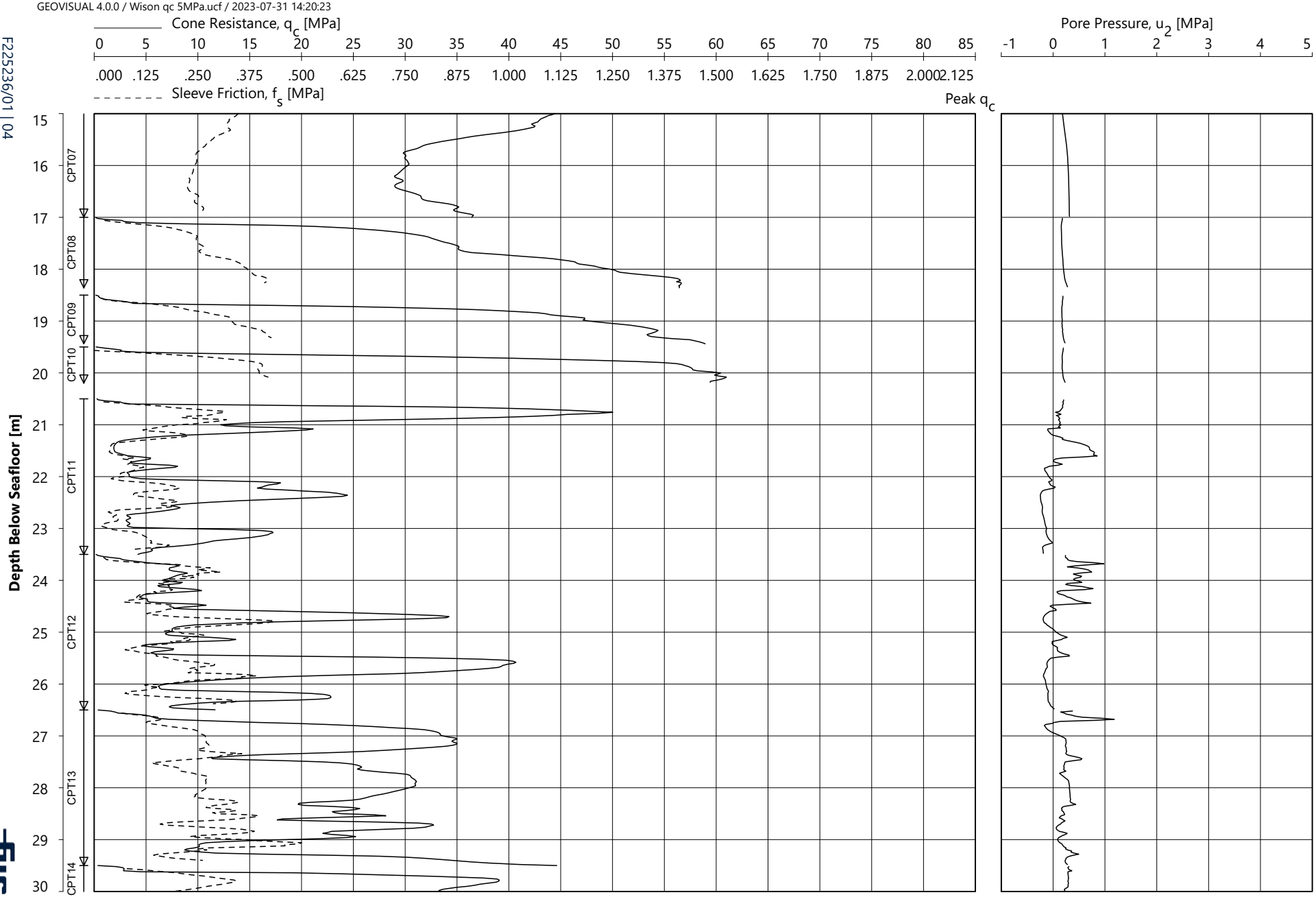

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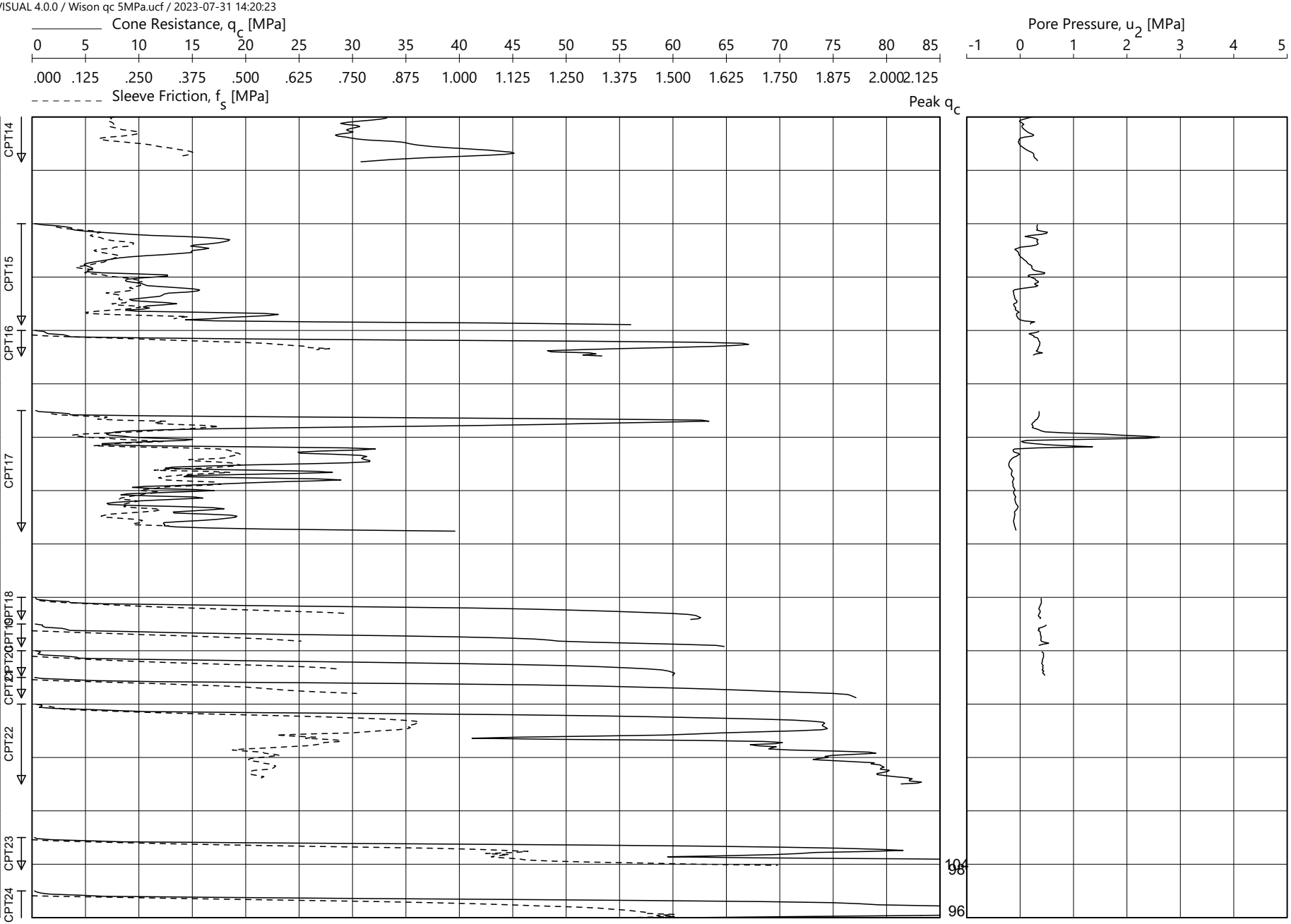

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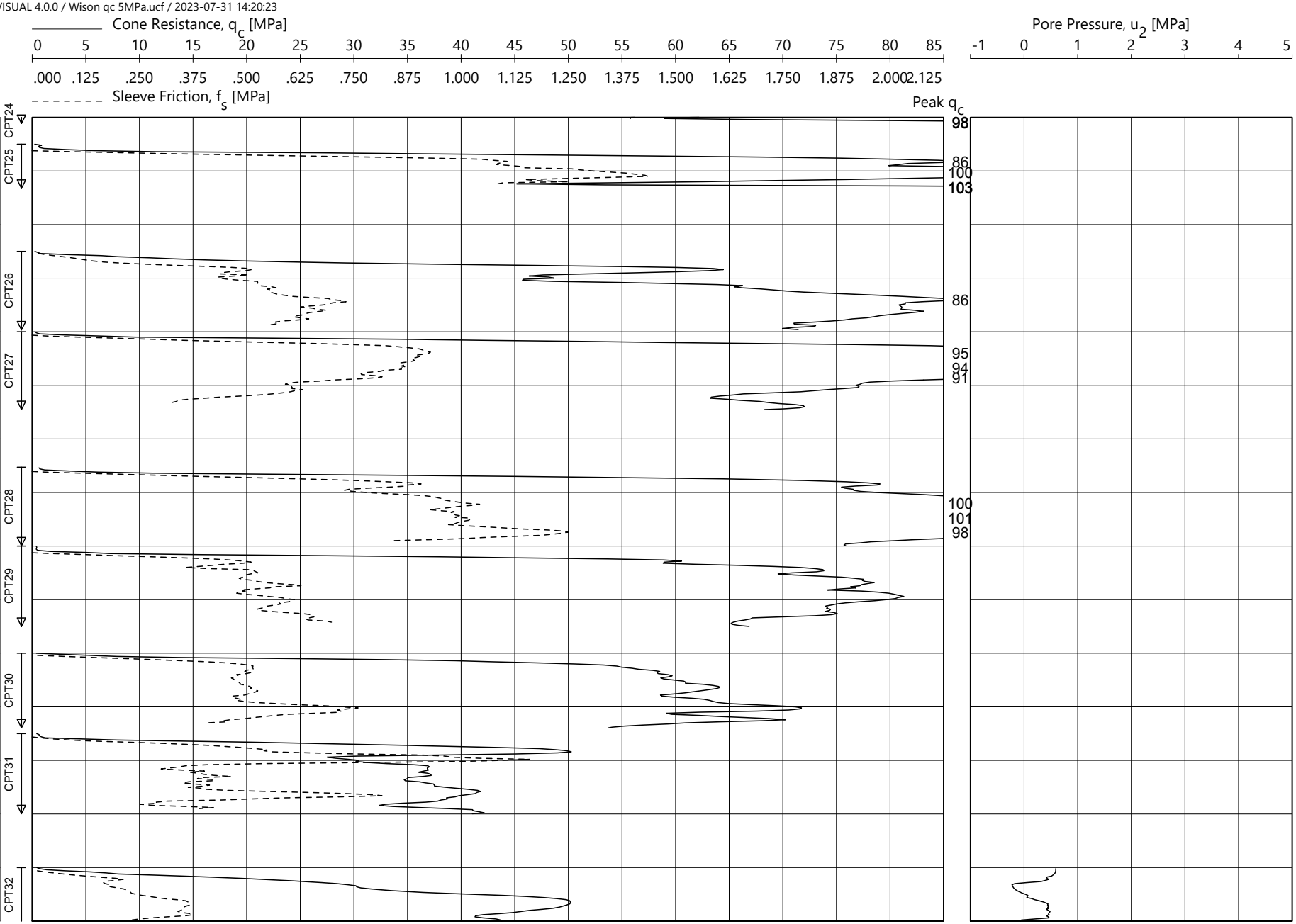

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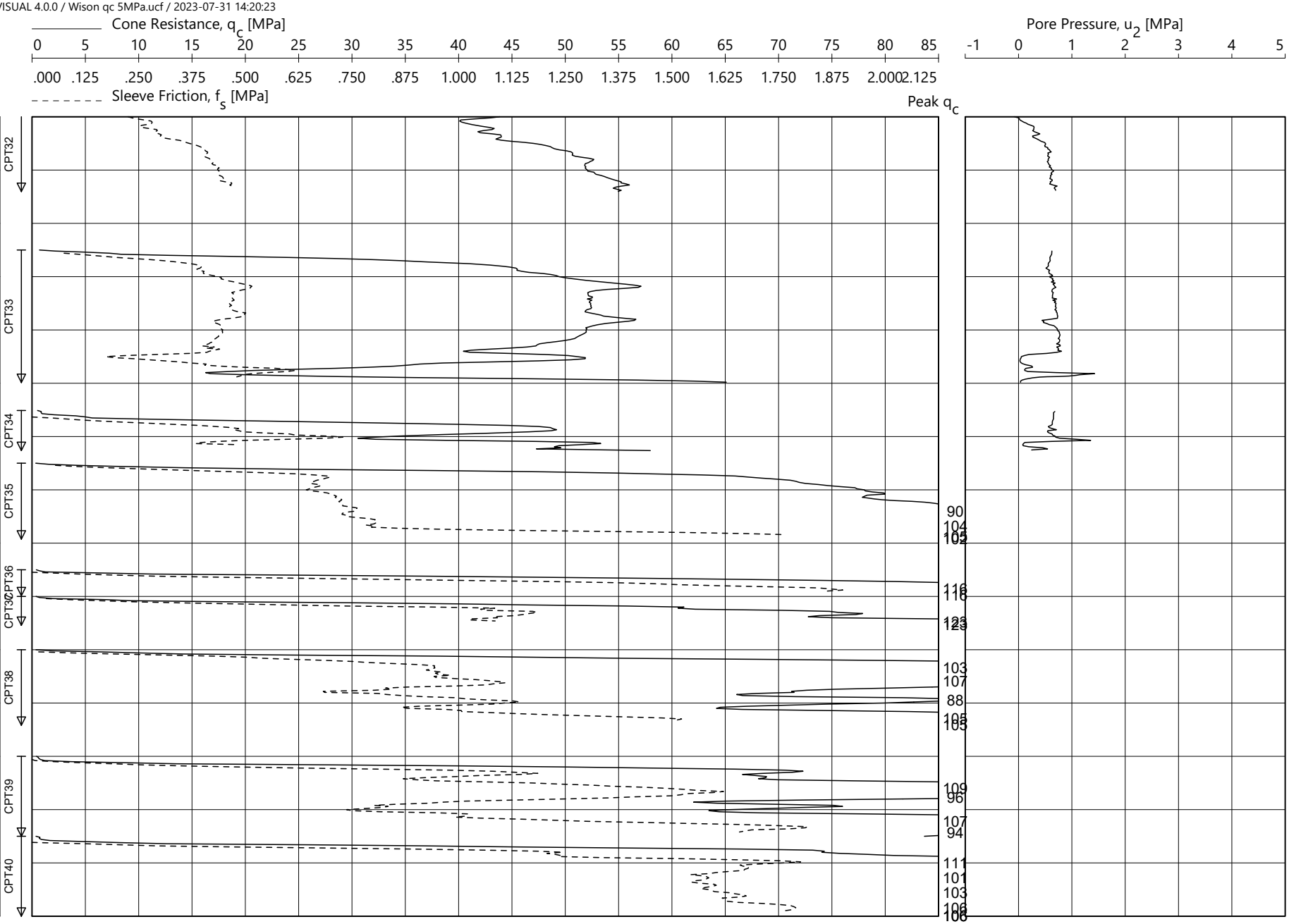

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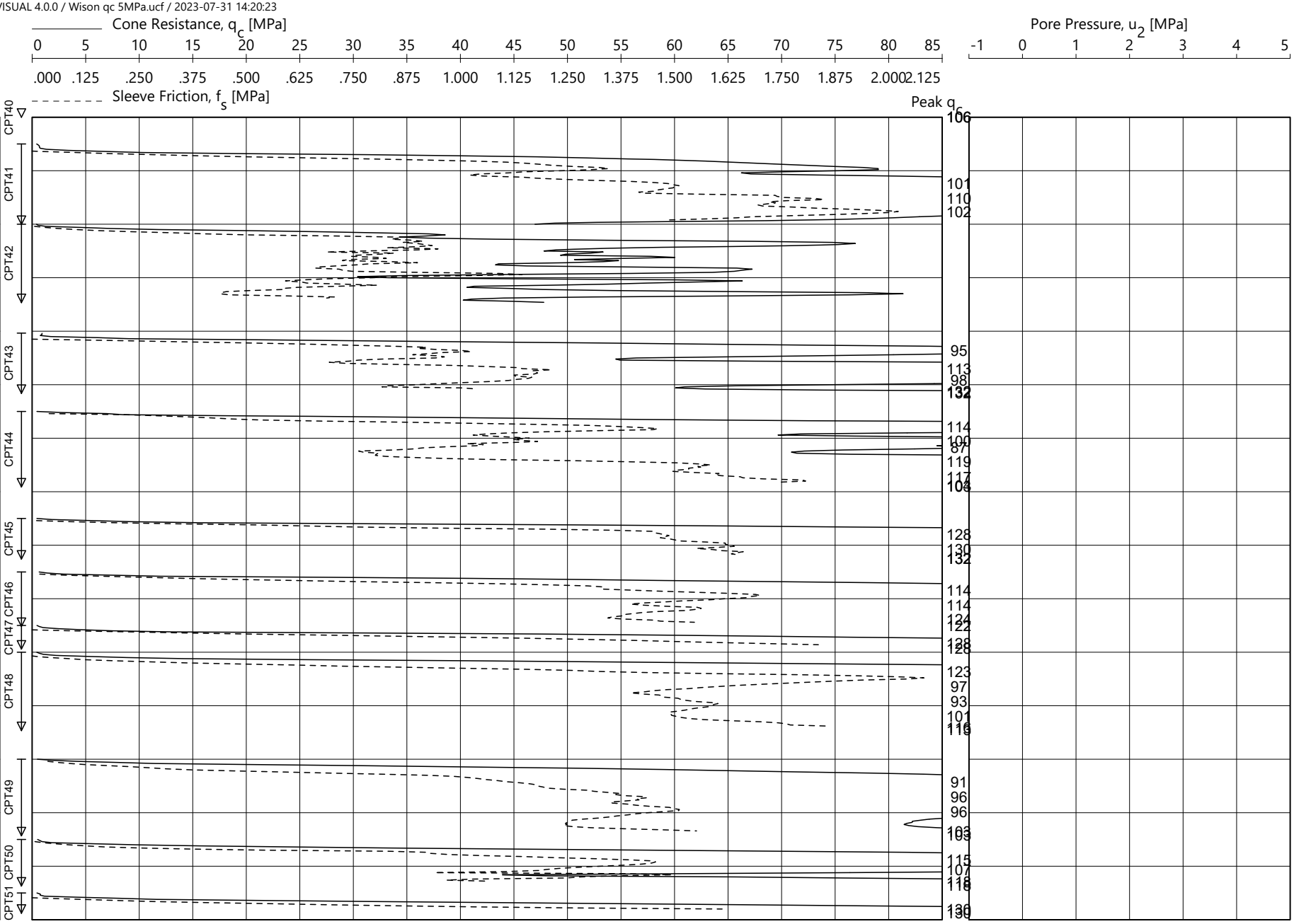

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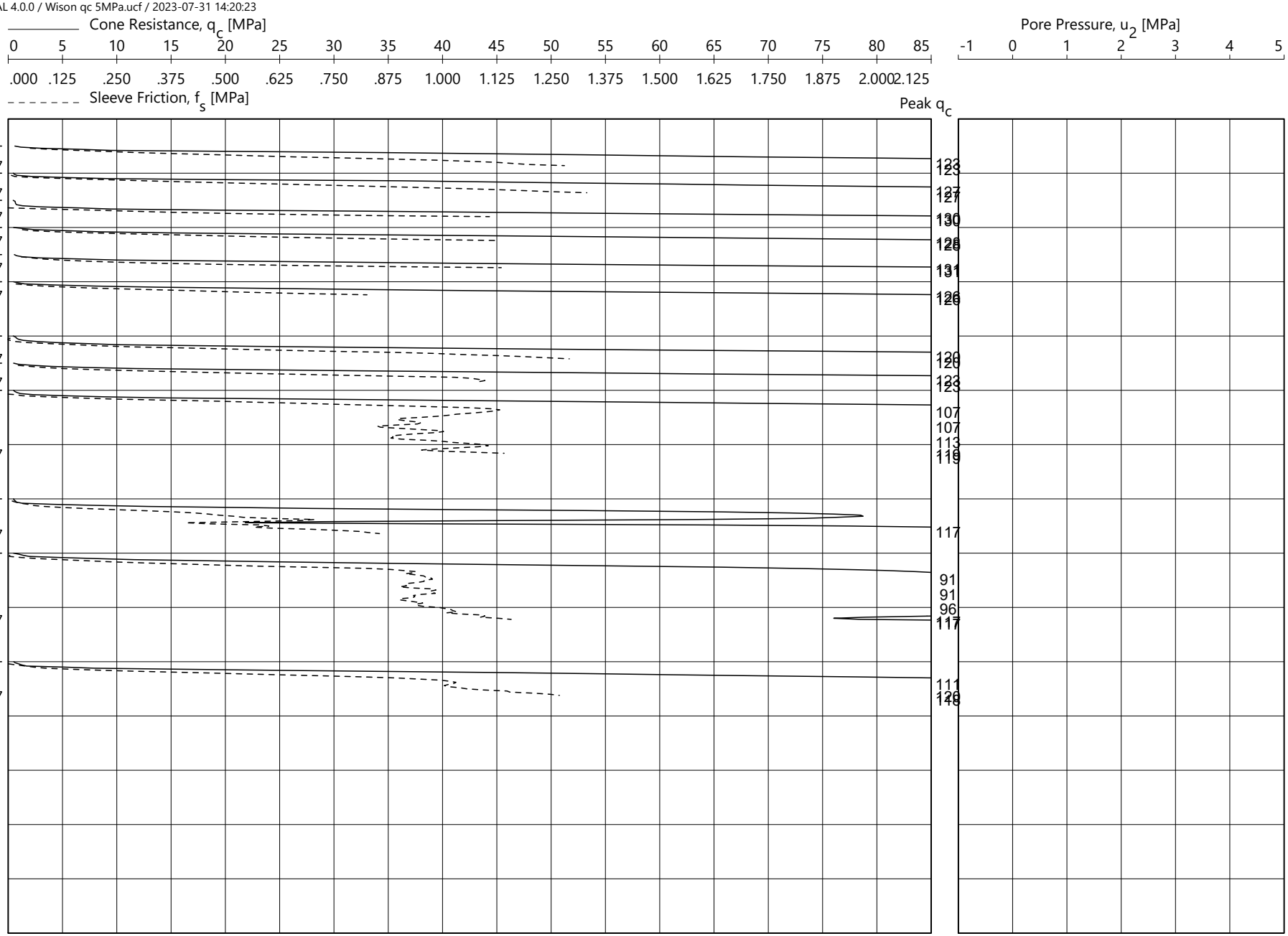

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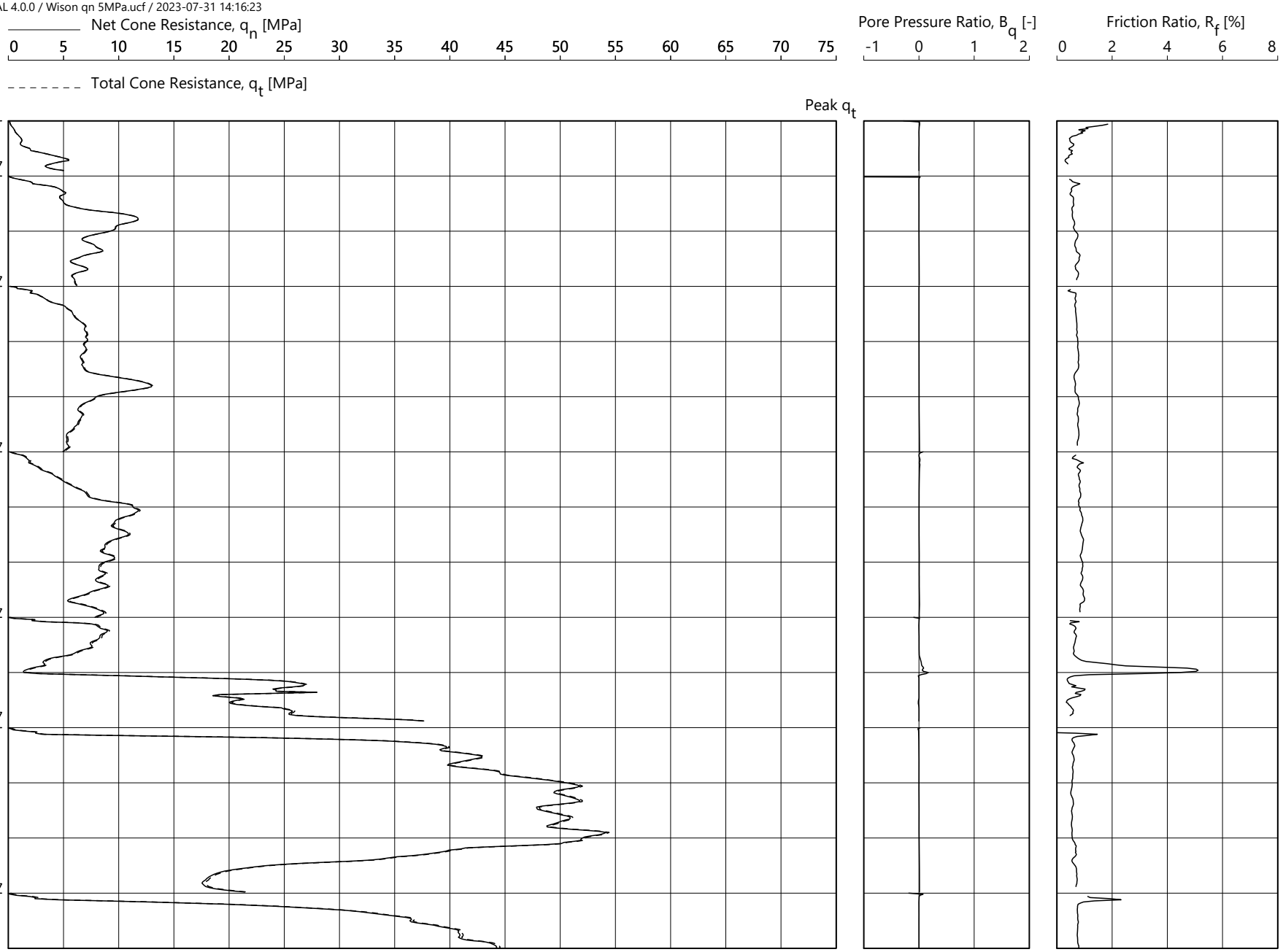

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Coordinates [m] : E 586494 N 5931306

CONE PENETRATION TEST
NEPL7-BH01

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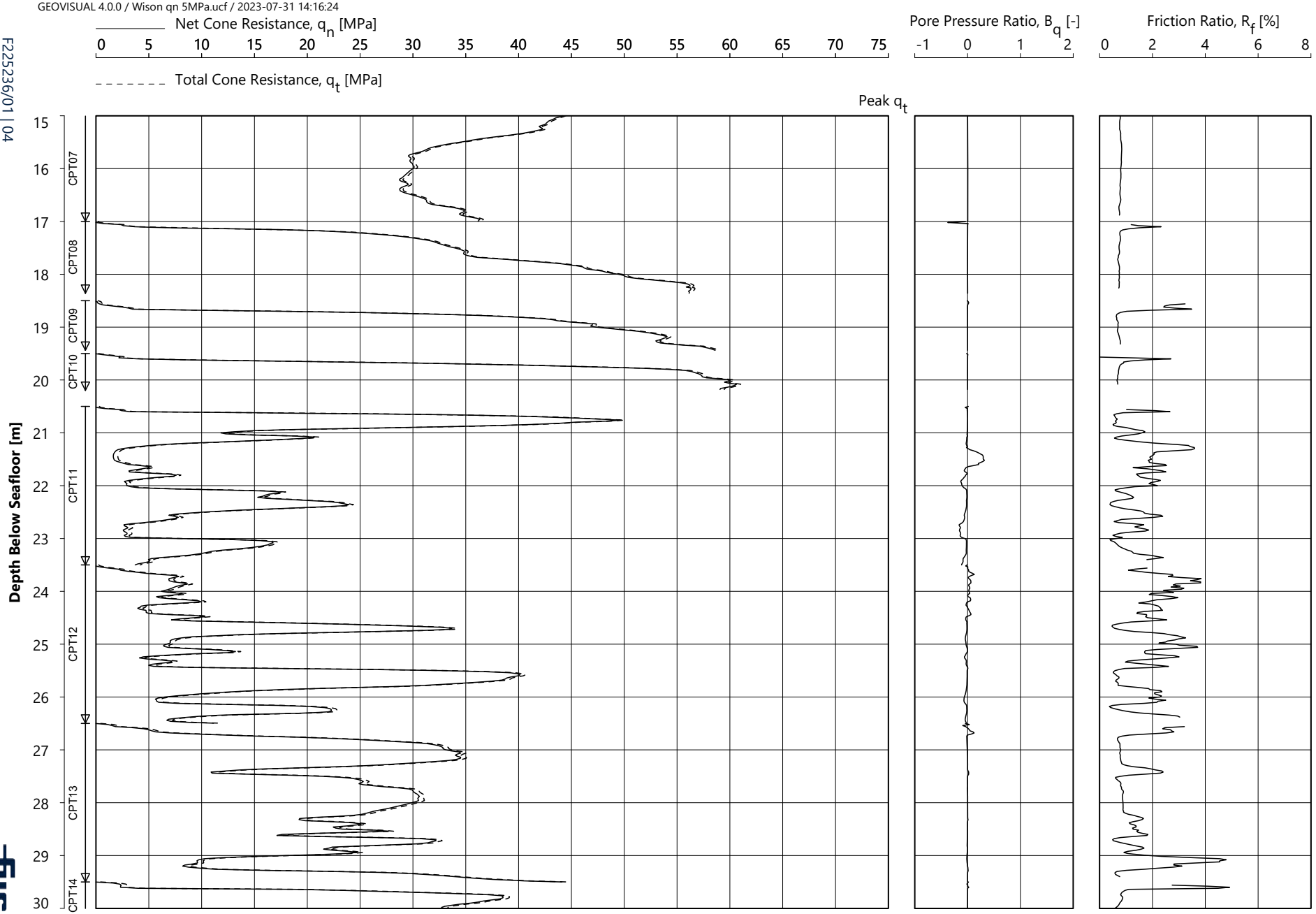

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CONE PENETRATION TEST
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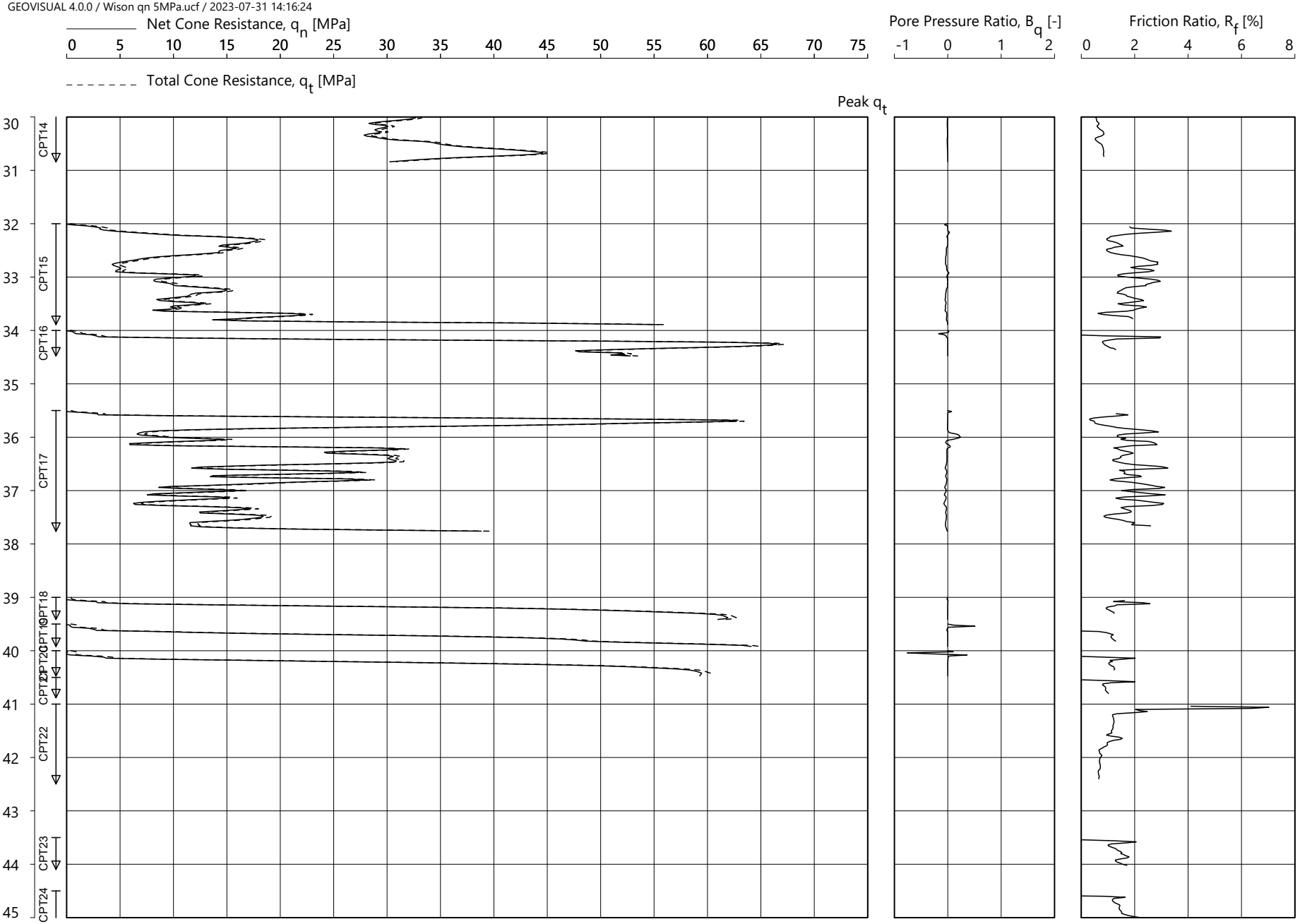

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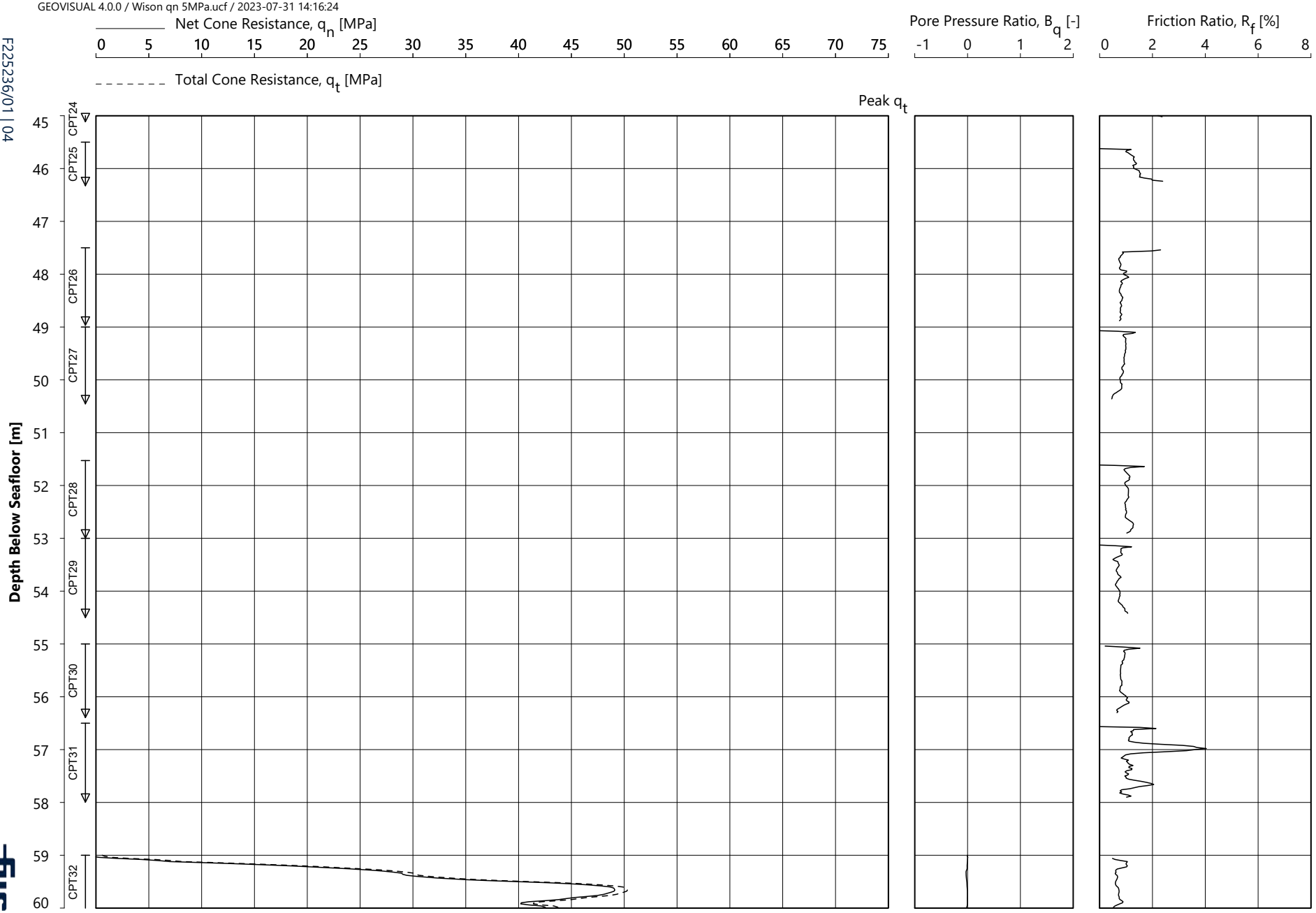

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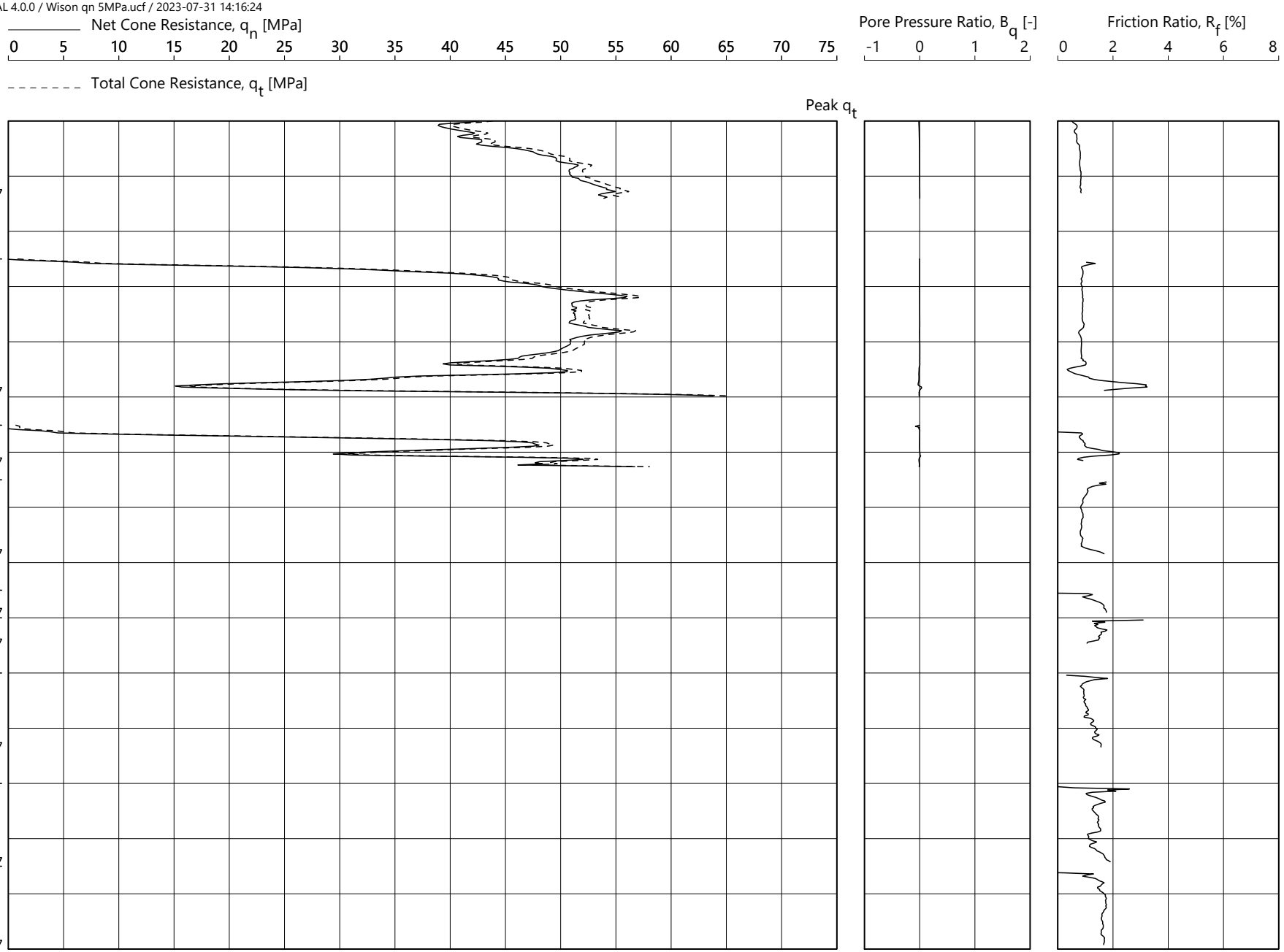
Coordinates [m] : E 586494 N 5931306

CONE PENETRATION TEST
NEPL7-BH01

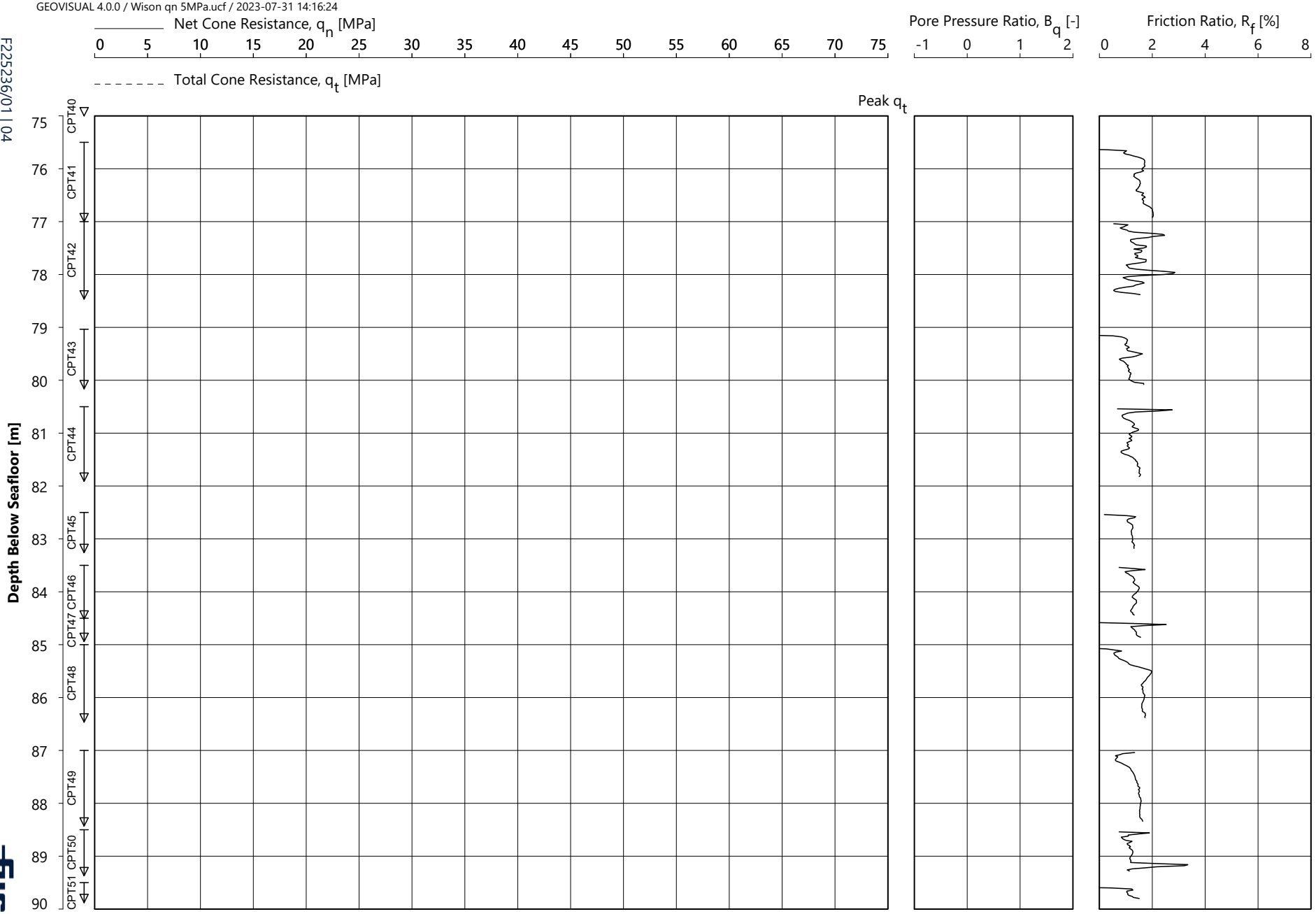
GEOVISUAL 4.0.0 / Wison qn 5MPa.ucf / 2023-07-31 14:16:24



Date Of Testing : 27-Feb-2023
Water Depth [m LAT] : 25.4
Coordinates [m] : E 586494 N 5931306

CONE PENETRATION TEST
NEPL7-BH01

GEOVISUAL 4.0.0 / Wison qn 5MPa.ucf / 2023-07-31 14:16:24



Date Of Testing : 27-Feb-2023
Water Depth [m LAT] : 25.4
Coordinates [m] : E 586494 N 5931306

CONE PENETRATION TEST
NEPL7-BH01

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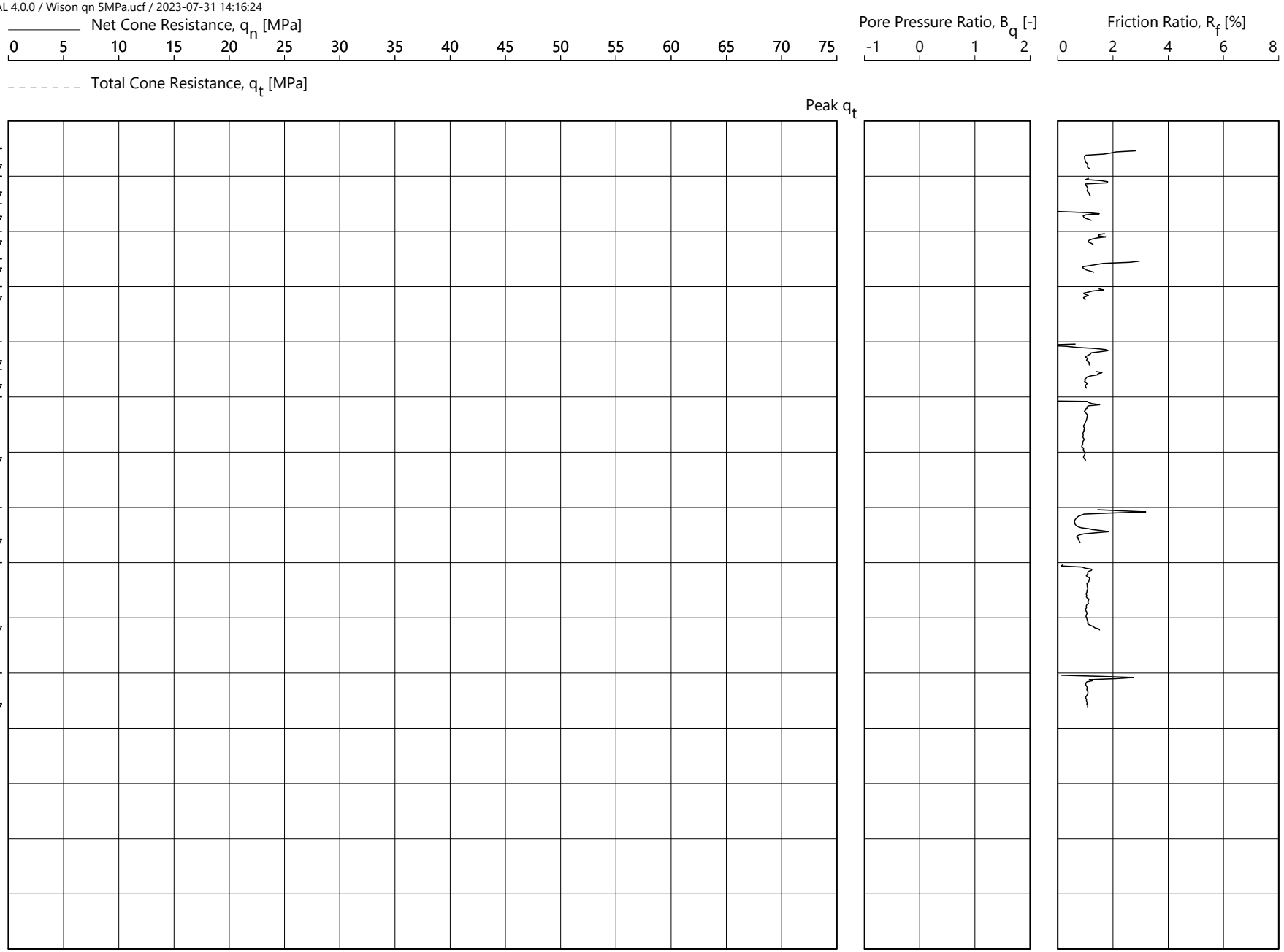
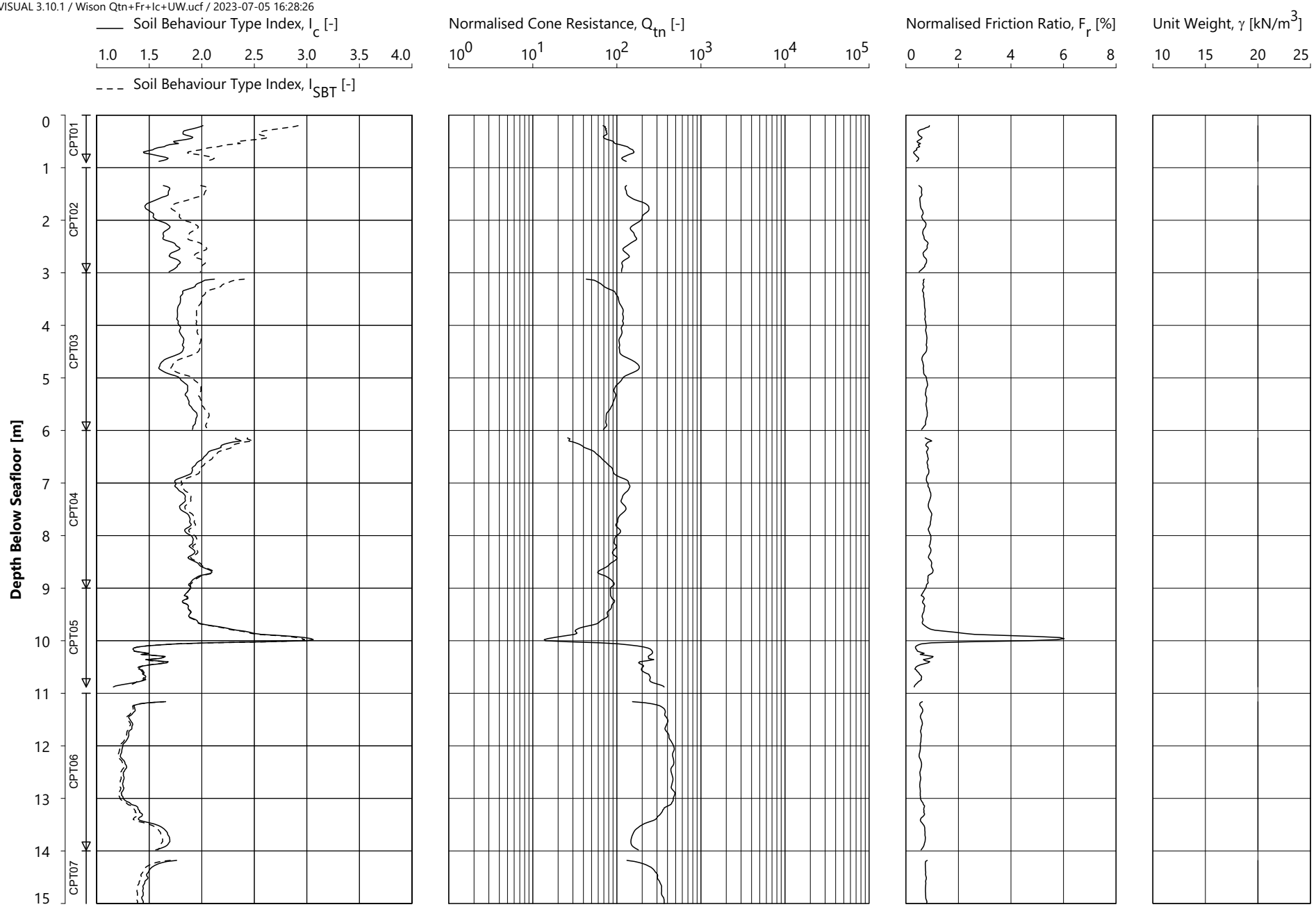


Date Of Testing : 27-Feb-2023
Water Depth [m LAT] : 25.4
Coordinates [m] : E 586494 N 5931306

CONE PENETRATION TEST
NEPL7-BH01

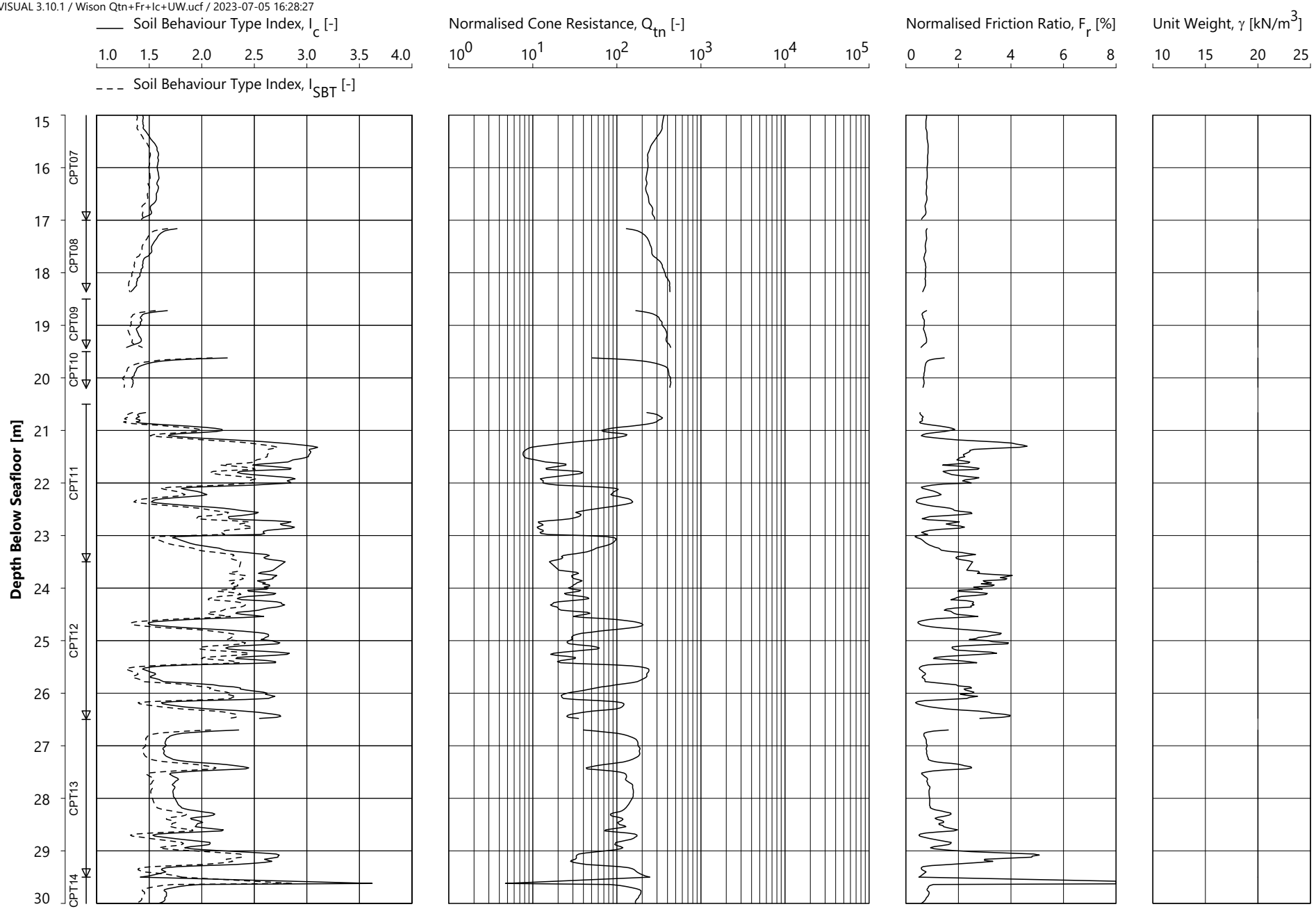


Date of Testing : 26-Feb-2023
Water Depth [m LAT] : 25.4
Coordinates [m] : E 586494 N 5931306

Probe Type : CP10-CF80PB20
Cone Base Area [mm^2] : 1007
Ground Water [kN/m^3] : 10.0

CONE PENETRATION TEST

NEPL7-BH01

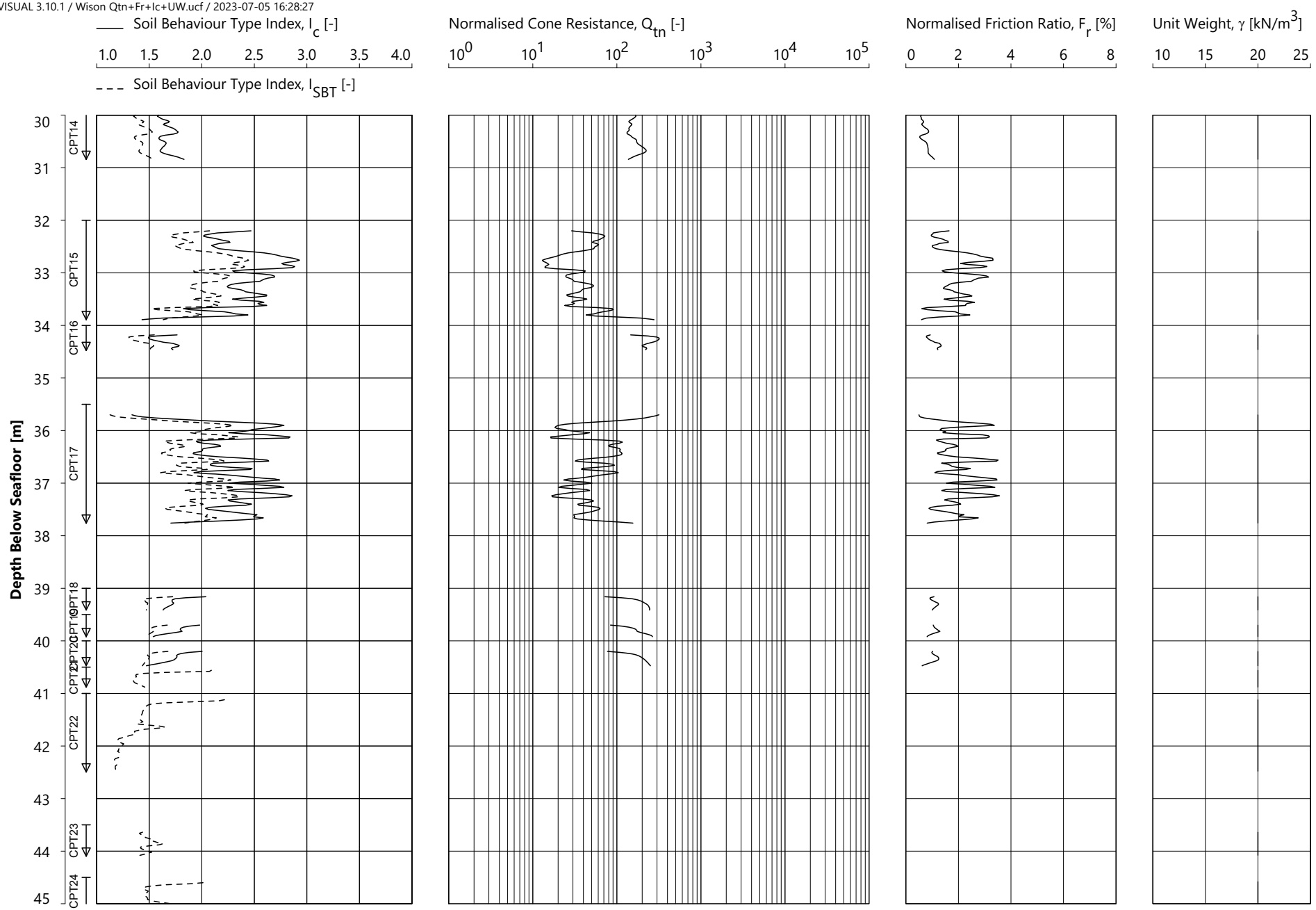


Date of Testing : 27-Feb-2023
Water Depth [m LAT] : 25.4
Coordinates [m] : E 586494 N 5931306

Probe Type : CP10-CF80PB20
Cone Base Area [mm^2] : 1007
Ground Water [kN/m^3] : 10.0

CONE PENETRATION TEST

NEPL7-BH01

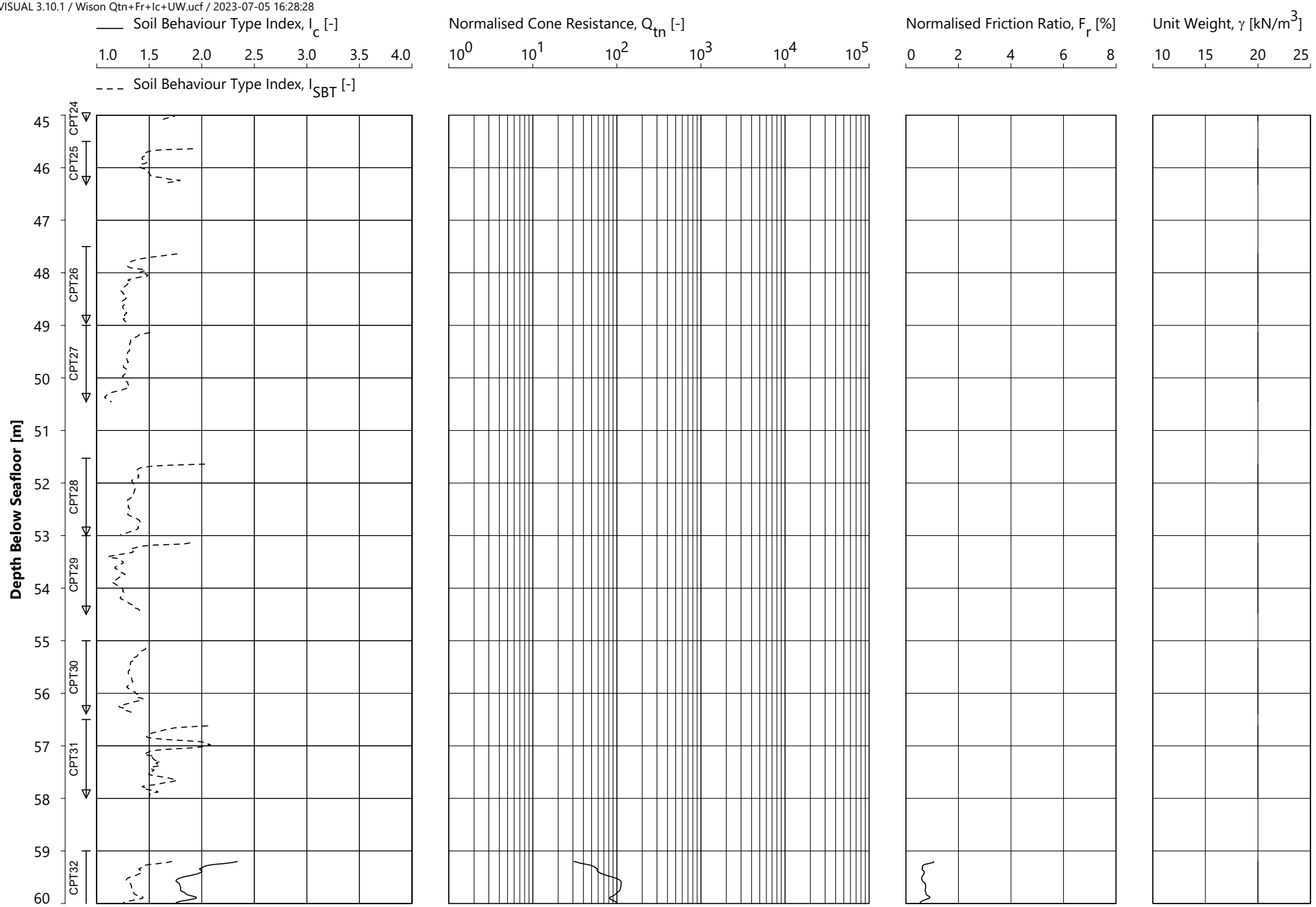


Date of Testing : 27-Feb-2023
Water Depth [m LAT] : 25.4
Coordinates [m] : E 586494 N 5931306

Probe Type : CP10-CF80PB20
Cone Base Area [mm^2] : 1007
Ground Water [kN/m^3] : 10.0

CONE PENETRATION TEST

NEPL7-BH01

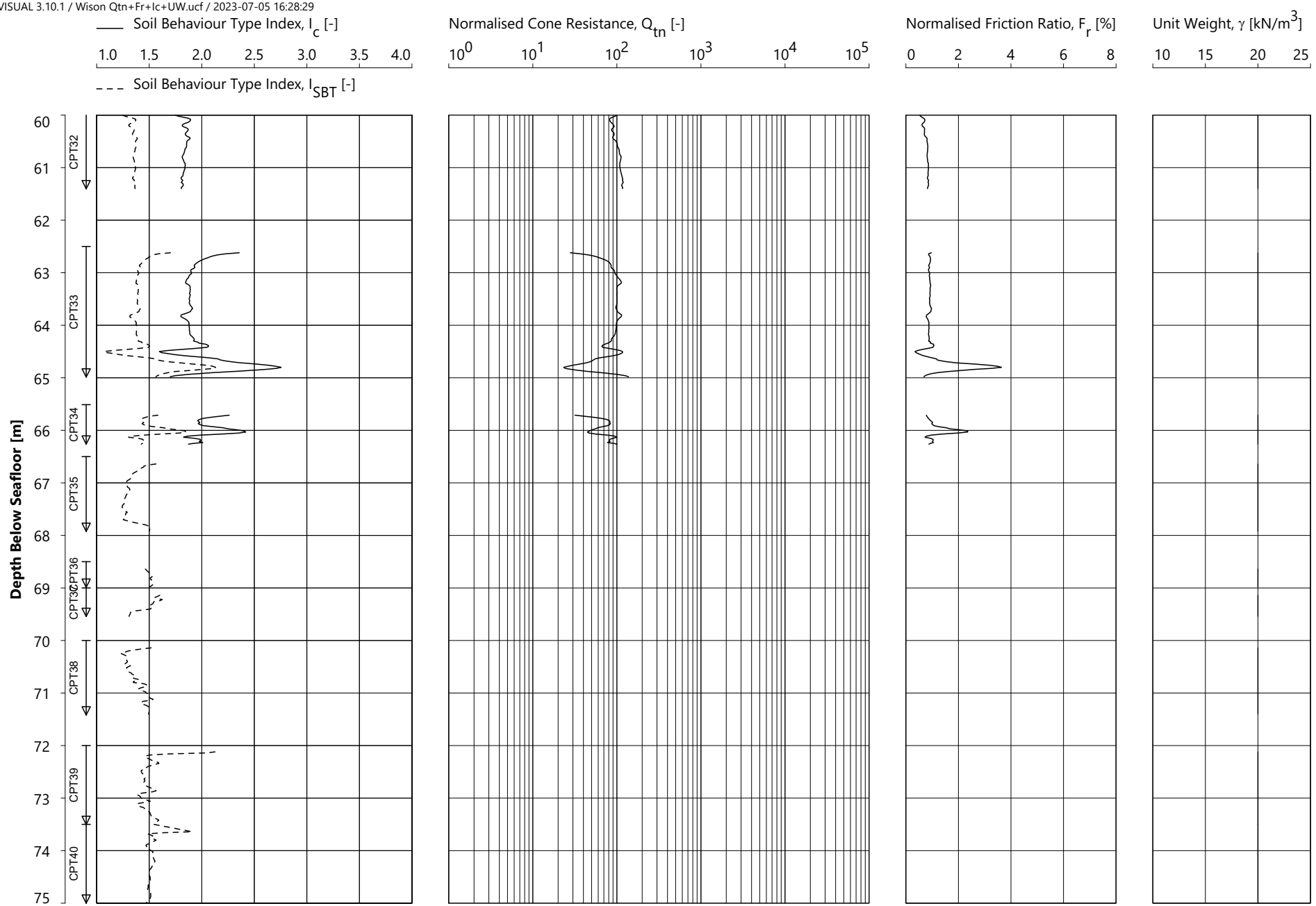


Date of Testing : 27-Feb-2023
Water Depth [m LAT] : 25.4
Coordinates [m] : E 586494 N 5931306

Probe Type : CP5-CF50
Cone Base Area [mm^2] : 503
Ground Water [kN/m^3] : 10.0

CONE PENETRATION TEST

NEPL7-BH01

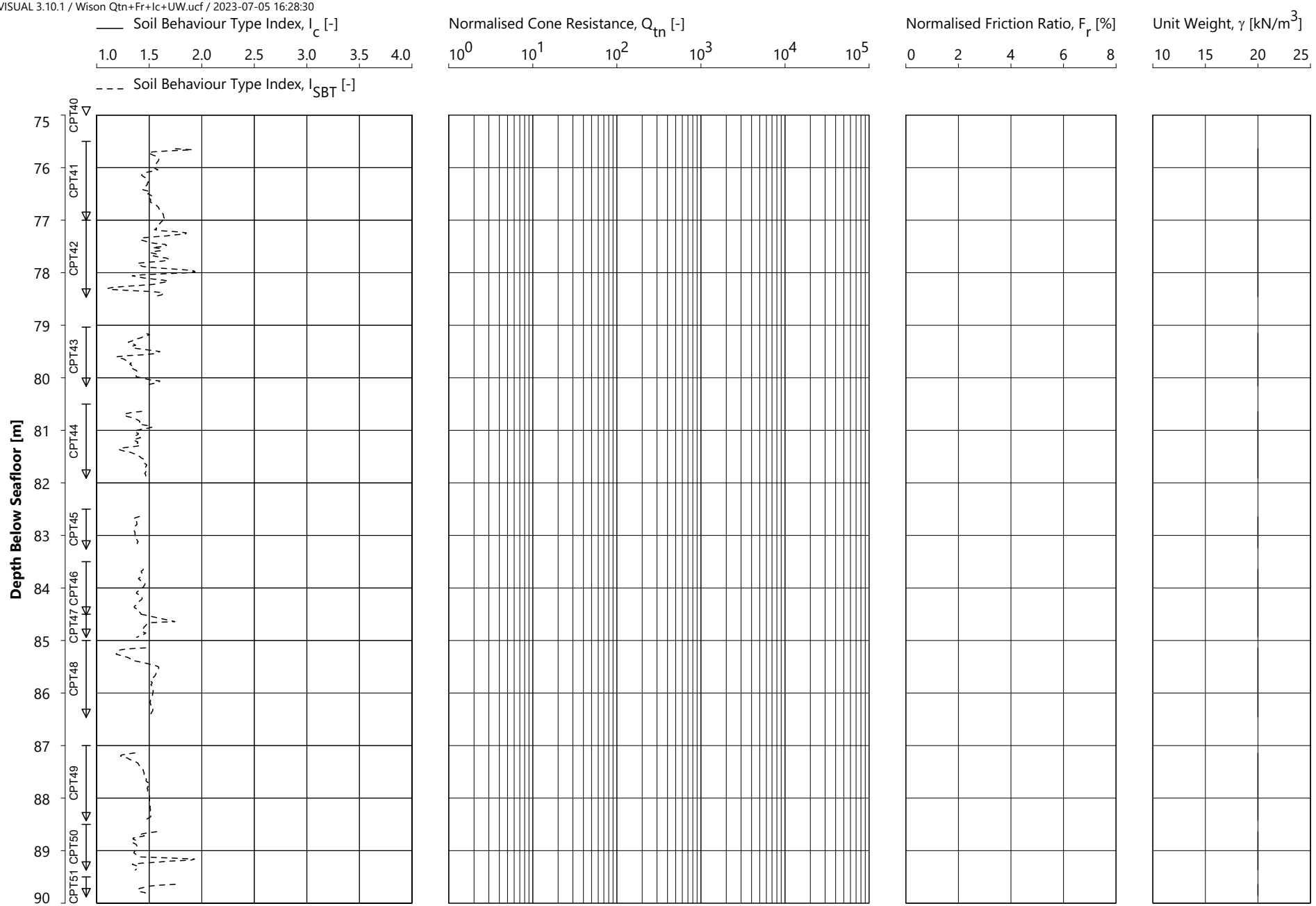


Date of Testing : 27-Feb-2023
Water Depth [m LAT] : 25.4
Coordinates [m] : E 586494 N 5931306

Probe Type : CP10-CF80PB20
Cone Base Area [mm^2] : 1007
Ground Water [kN/m^3] : 10.0

CONE PENETRATION TEST

NEPL7-BH01

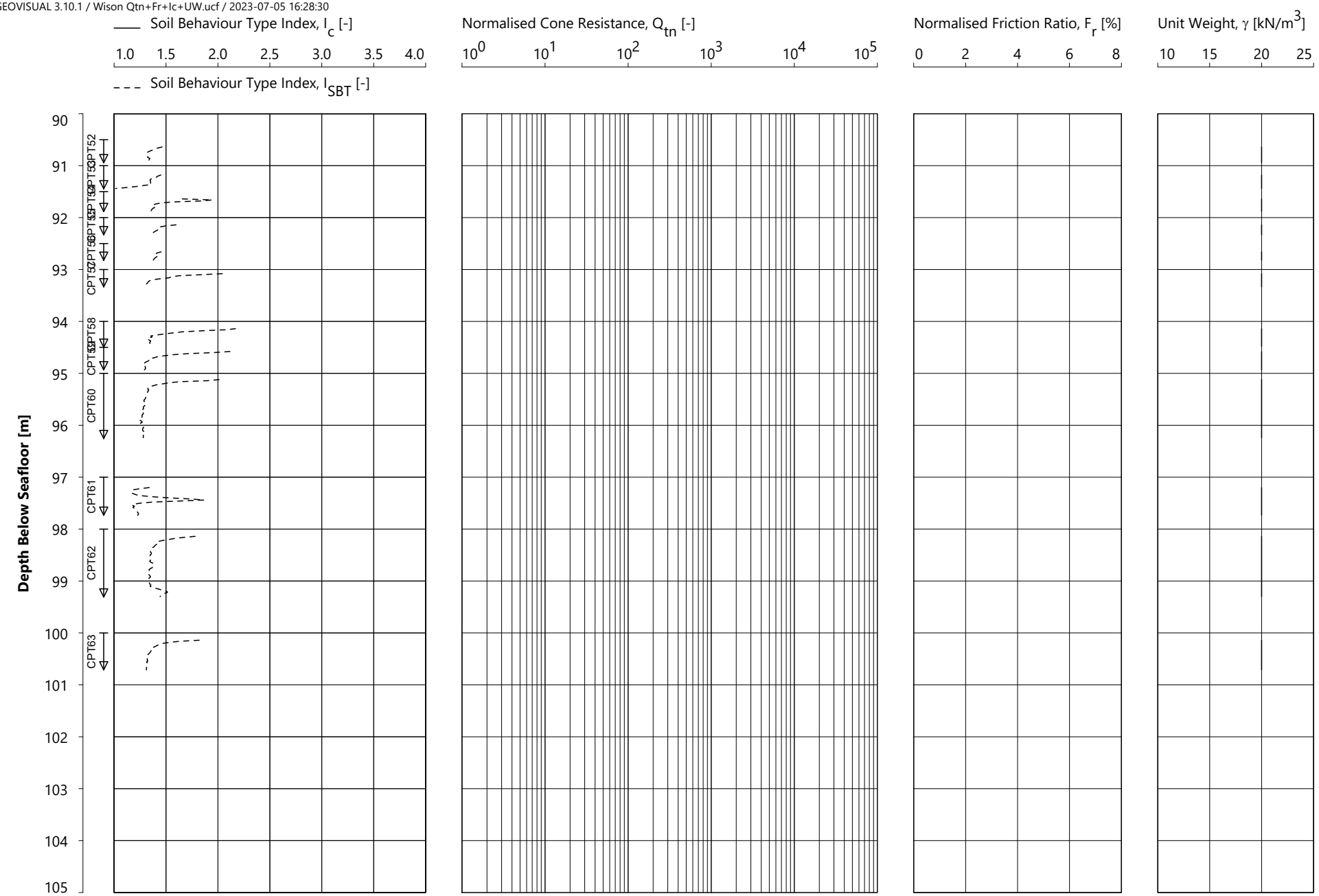


Date of Testing : 27-Feb-2023
Water Depth [m LAT] : 25.4
Coordinates [m] : E 586494 N 5931306

Probe Type : CP5-CF50
Cone Base Area [mm^2] : 503
Ground Water [kN/m^3] : 10.0

CONE PENETRATION TEST

NEPL7-BH01



Date of Testing : 27-Feb-2023
Water Depth [m LAT] : 25.4
Coordinates [m] : E 586494 N 5931306

Probe Type : CP5-CF50
Cone Base Area [mm^2] : 503
Ground Water [kN/m^3] : 10.0

CONE PENETRATION TEST

NEPL7-BH01

Cone Penetration Test - Zero Drift

Borehole		Zero Reading at Start of Test			Zero Drift			Probe		Net Area Ratio
		q_c [MPa]	f_s [MPa]	u [MPa]	q_c [MPa]	f_s [MPa]	u [MPa]			a [-]
NEPL7-BH01	CPT01	0.025	-0.005	-0.043	0.014	0.001	0.004	CP10-CF80PB20	1706-2485	0.750
NEPL7-BH01	CPT02	0.036	-0.004	-0.042	-0.021	0.000	0.007	CP10-CF80PB20	1706-2485	0.750
NEPL7-BH01	CPT03	0.032	-0.005	-0.041	-0.040	0.001	0.006	CP10-CF80PB20	1706-2485	0.750
NEPL7-BH01	CPT04	0.040	-0.005	-0.019	-0.058	0.001	-0.020	CP10-CF80PB20	1706-2485	0.750
NEPL7-BH01	CPT05	0.044	-0.005	-0.068	0.011	0.002	0.006	CP10-CF80PB20	1706-2486	0.750
NEPL7-BH01	CPT06	0.066	-0.003	-0.076	-0.056	0.001	0.022	CP10-CF80PB20	1706-2486	0.750
NEPL7-BH01	CPT07	0.066	-0.003	-0.072	-0.047	0.000	0.014	CP10-CF80PB20	1706-2486	0.750
NEPL7-BH01	CPT08	0.067	-0.003	-0.075	-0.028	0.003	0.009	CP10-CF80PB20	1706-2486	0.750
NEPL7-BH01	CPT09	0.049	-0.001	-0.077	-0.025	-0.001	0.006	CP10-CF80PB20	1706-2486	0.750
NEPL7-BH01	CPT10	0.069	-0.002	-0.076	-0.025	0.001	0.011	CP10-CF80PB20	1706-2486	0.750
NEPL7-BH01	CPT11	0.063	-0.003	-0.079	-0.003	0.002	0.012	CP10-CF80PB20	1706-2486	0.750
NEPL7-BH01	CPT12	0.065	-0.002	-0.073	-0.025	0.001	0.008	CP10-CF80PB20	1706-2486	0.750
NEPL7-BH01	CPT13	0.057	-0.002	-0.074	-0.037	0.001	0.011	CP10-CF80PB20	1706-2486	0.750
NEPL7-BH01	CPT14	0.049	-0.001	-0.076	-0.011	0.000	0.014	CP10-CF80PB20	1706-2486	0.750
NEPL7-BH01	CPT15	0.063	-0.001	-0.080	0.018	-0.001	0.012	CP10-CF80PB20	1706-2486	0.750
NEPL7-BH01	CPT16	0.069	-0.002	-0.078	-0.024	0.000	-0.003	CP10-CF80PB20	1706-2486	0.750
NEPL7-BH01	CPT17	0.079	-0.003	-0.084	-0.033	0.003	0.012	CP10-CF80PB20	1706-2486	0.750
NEPL7-BH01	CPT18	0.073	-0.002	-0.084	-0.013	0.002	0.008	CP10-CF80PB20	1706-2486	0.750
NEPL7-BH01	CPT19	0.061	-0.001	-0.084	-0.069	0.003	0.007	CP10-CF80PB20	1706-2486	0.750
NEPL7-BH01	CPT20	0.007	0.002	-0.082	0.025	-0.003	0.011	CP10-CF80PB20	1706-2486	0.750
NEPL7-BH01	CPT21	-0.053	0.000		-0.019	0.000		CP5-CF50	1709-0191	0.500
NEPL7-BH01	CPT22	-0.080	0.000		0.021	0.001		CP5-CF50	1709-0191	0.500
NEPL7-BH01	CPT23	-0.071	0.003		-0.016	-0.005		CP5-CF50	1709-0191	0.500
NEPL7-BH01	CPT24	-0.064	0.003		-0.035	-0.019		CP5-CF50	1709-0191	0.500
NEPL7-BH01	CPT25	-0.071	-0.005		-0.010	0.006		CP5-CF50	1709-0191	0.500

Key:

q_c : cone resistance f_s : sleeve friction u : pore water pressure

Note:

1. Zero Drift is the difference between the zero output at the start of the test and the zero output at the end of the test. Offshore tests may show Reference Readings. The Zero Reading or Reference Reading at Start of Test is a value presented in units of measurement result. The value itself is a conversion from system output, usually in mV or in bits. It has no explicit physical meaning.
2. --- : Zero Drift was not monitored. The drift can be assessed from the start values of successive tests.

Cone Penetration Test - Zero Drift

Borehole		Zero Reading at Start of Test			Zero Drift			Probe		Net Area Ratio
		q_c [MPa]	f_s [MPa]	u [MPa]	q_c [MPa]	f_s [MPa]	u [MPa]			a [-]
NEPL7-BH01	CPT26	-0.085	0.001		-0.005	0.001		CP5-CF50	1709-0191	0.500
NEPL7-BH01	CPT27	-0.058	0.001		-0.002	-0.002		CP5-CF50	1709-0191	0.500
NEPL7-BH01	CPT28	-0.094	0.000		-0.018	0.001		CP5-CF50	1709-0191	0.500
NEPL7-BH01	CPT29	-0.100	0.005		-0.007	-0.006		CP5-CF50	1709-0191	0.500
NEPL7-BH01	CPT30	-0.103	0.001		-0.013	0.002		CP5-CF50	1709-0191	0.500
NEPL7-BH01	CPT31	-0.109	0.003		-0.006	0.001		CP5-CF50	1709-0191	0.500
NEPL7-BH01	CPT32	0.004	0.002	-0.065	0.000	-0.003	0.018	CP10-CF80PB20	1706-2486	0.750
NEPL7-BH01	CPT33	0.063	-0.002	-0.068	-0.017	0.001	0.016	CP10-CF80PB20	1706-2486	0.750
NEPL7-BH01	CPT34	0.067	-0.003	-0.069	-0.038	0.002	0.011	CP10-CF80PB20	1706-2486	0.750
NEPL7-BH01	CPT35	-0.113	0.003		-0.019	-0.001		CP5-CF50	1709-0191	0.500
NEPL7-BH01	CPT36	-0.110	0.003		-0.013	-0.002		CP5-CF50	1709-0191	0.500
NEPL7-BH01	CPT37	-0.115	0.002		0.001	0.003		CP5-CF50	1709-0191	0.500
NEPL7-BH01	CPT38	-0.117	0.005		-0.015	-0.001		CP5-CF50	1709-0191	0.500
NEPL7-BH01	CPT39	-0.111	0.005		-0.014	0.001		CP5-CF50	1709-0191	0.500
NEPL7-BH01	CPT40	-0.117	0.007		-0.014	-0.001		CP5-CF50	1709-0191	0.500
NEPL7-BH01	CPT41	-0.114	0.005		-0.039	-0.007		CP5-CF50	1709-0191	0.500
NEPL7-BH01	CPT42	-0.128	0.005		-0.010	0.005		CP5-CF50	1709-0191	0.500
NEPL7-BH01	CPT43	-0.124	0.008		-0.023	-0.003		CP5-CF50	1709-0191	0.500
NEPL7-BH01	CPT44	-0.129	0.007		-0.024	-0.007		CP5-CF50	1709-0191	0.500
NEPL7-BH01	CPT45	-0.129	0.003		-0.017	0.006		CP5-CF50	1709-0191	0.500
NEPL7-BH01	CPT46	-0.135	0.011		-0.013	-0.005		CP5-CF50	1709-0191	0.500
NEPL7-BH01	CPT47	-0.127	0.009		-0.370	0.019		CP5-CF50	1709-0191	0.500
NEPL7-BH01	CPT48	0.016	0.059		0.432	-0.040		CP5-CF50	1709-0055	0.500
NEPL7-BH01	CPT49	0.001	-0.003		-0.066	-0.002		CP5-CF50	1721-1872	0.500
NEPL7-BH01	CPT50	-0.043	-0.005		-0.013	-0.001		CP5-CF50	1721-1872	0.500

Key:

q_c : cone resistance f_s : sleeve friction u : pore water pressure

Note:

1. Zero Drift is the difference between the zero output at the start of the test and the zero output at the end of the test. Offshore tests may show Reference Readings. The Zero Reading or Reference Reading at Start of Test is a value presented in units of measurement result. The value itself is a conversion from system output, usually in mV or in bits. It has no explicit physical meaning.
2. --- : Zero Drift was not monitored. The drift can be assessed from the start values of successive tests.

Cone Penetration Test - Zero Drift

Borehole		Zero Reading at Start of Test			Zero Drift			Probe		Net Area Ratio
		q_c [MPa]	f_s [MPa]	u [MPa]	q_c [MPa]	f_s [MPa]	u [MPa]			a [-]
NEPL7-BH01	CPT51	-0.051	-0.005		-0.001	0.001		CP5-CF50	1721-1872	0.500
NEPL7-BH01	CPT52	-0.047	-0.002		-0.005	-0.002		CP5-CF50	1721-1872	0.500
NEPL7-BH01	CPT53	-0.040	0.000		0.012	-0.003		CP5-CF50	1721-1872	0.500
NEPL7-BH01	CPT54	-0.041	0.003		-0.010	-0.006		CP5-CF50	1721-1872	0.500
NEPL7-BH01	CPT55	-0.050	0.001		0.004	-0.004		CP5-CF50	1721-1872	0.500
NEPL7-BH01	CPT56	-0.036	0.000		---	---		CP5-CF50	1721-1872	0.500
NEPL7-BH01	CPT57	-0.038	-0.002		-0.002	0.000		CP5-CF50	1721-1872	0.500
NEPL7-BH01	CPT58	-0.043	0.001		-0.003	-0.003		CP5-CF50	1721-1872	0.500
NEPL7-BH01	CPT59	-0.041	0.010		-0.009	-0.013		CP5-CF50	1721-1872	0.500
NEPL7-BH01	CPT60	-0.036	0.004		-0.035	-0.008		CP5-CF50	1721-1872	0.500
NEPL7-BH01	CPT61	-0.048	-0.002		-0.001	-0.001		CP5-CF50	1721-1872	0.500
NEPL7-BH01	CPT62	-0.032	0.005		-0.030	-0.009		CP5-CF50	1721-1872	0.500
NEPL7-BH01	CPT63	-0.054	0.001		-0.012	-0.005		CP5-CF50	1721-1872	0.500

Key:

q_c : cone resistance f_s : sleeve friction u : pore water pressure

Note:

1. Zero Drift is the difference between the zero output at the start of the test and the zero output at the end of the test. Offshore tests may show Reference Readings. The Zero Reading or Reference Reading at Start of Test is a value presented in units of measurement result. The value itself is a conversion from system output, usually in mV or in bits. It has no explicit physical meaning.
2. --- : Zero Drift was not monitored. The drift can be assessed from the start values of successive tests.

Appendix D

Geotechnical Laboratory Test Results

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D.1 Laboratory Testing Overview

D.1.1 Practice for Sample Handling

General Practice	
General Procedure:	■ According to ISO 19901-8:2014 ■ Refer to document titled 'Geotechnical Laboratory Tests' presented in the appendix titled 'Descriptions of Methods and Practices'
Information Management:	■ Laboratory-specific software and GeODin® software, as applicable ■ PDF for viewing and printing, where applicable (this primary document)

Initial Sample Handling	
Tube Sample:	■ On-deck removal of sample tube from sampler ■ Measurement of recovery ■ Transfer of sample tube to site laboratory

Site Geotechnical Laboratory	
Laboratory Air Temperature:	Typically about 20 °C
Geotechnical Sample in Tube:	■ Sample extrusion ■ Sample photography ■ Laboratory tests in accordance with Main Text and document section titled 'Practice for Geotechnical Laboratory Tests' ■ Selection and labelling of geotechnical sample sections for undisturbed preservation and/or disturbed preservation

Sample Protection – Packaging	
Selected Sample Sections:	■ Undisturbed sample sections wrapped in aluminium and/or plastic foil, and waxed into labelled cardboard tubes; cardboard tubes put into inflated, plastic protective bag ■ Disturbed sample sections placed in double set of plastic bags, and labelled ■ Placement in labelled shipping container

Sample Protection – Onsite Storage	
Geotechnical Sample:	■ Onboard storage area ■ Active temperature control within the range +4 °C and +7 °C ■ Protection from direct sunlight

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Sample Transport	
Fugro Sample Transport:	▪ Offloading of shipping containers from vessel at port of demobilisation ▪ Road freight to Fugro-nominated laboratories
Client Sample Transport:	Not applicable

Office Geotechnical Laboratory	
Laboratory Air Temperature:	Typically about 20 °C
Sample in Plastic Bags:	▪ Laboratory tests in accordance with subsection titled 'Practice for Geotechnical Laboratory Tests' ▪ If applicable, selection and labelling of left-over sample sections for disturbed preservation

Sample Storage and Disposal	
Sample Storage:	▪ Storage of bag samples (prior and after testing) and sample material from undisturbed samples saved in bags after testing in a warehouse at ground level with temperature typically within the range between +5 °C and +20 °C. Warehouse is heated when appropriate, no specific measures are taken to avoid temperature exceeding +20 °C (warehouse temperature is recorded with sensors approximately 2 m above floor level). ▪ Protection from direct sunlight ▪ After laboratory test programme is completed, storage of samples at a long term storage facility where temperature is constant at around +14 °C throughout the year
Transport:	After laboratory test programme is completed, road-freight to long term storage facility
Final Disposal:	In accordance with office laboratory procedures

D.1.2 Practice for Geotechnical Laboratory Tests

Practice	
General Procedure:	▪ According to ISO 19901-8:2014 ▪ Refer to documents titled 'Geotechnical Laboratory Tests' and 'Soil Description and Classification' presented in the appendix titled 'Descriptions of Methods and Practices'
Sample Handling:	Refer to document section titled 'Practice for Sample Handling'
Data Processing and Management:	▪ Laboratory-specific software and GeODin® software, as applicable

	▪ Graphical scales selected to suit general presentation of data ▪ No display of data outside of chart limits, i.e. some values may not be shown
Data Format(s):	PDF for viewing and printing, primary deliverable

Site Geotechnical Laboratory	
Test Programme:	▪ According to project specifications and within applicability of test type/method ▪ ISO standards are primary, where applicable
Programme Adjustment:	Assessment of feasibility of a test (by sample inspection) and, if required, selection of alternative test specimen if adequate test completion proves impracticable
Ground Description:	▪ According to document titled 'Soil Description and Classification' presented in the appendix titled 'Descriptions of Methods and Practices', as applicable ▪ Based on ISO 14688-1:2017, ISO 14688-2:2017, and BS 5930:2015+A1:2020 ▪ Carbonate content designation according to ISO 14688-1:2017 ▪ Geotechnical sample description is an interpretation of processed data available and/or selected at the time of preparation ▪ No assessment of geological origin ▪ No update of geotechnical sample descriptions made after site geotechnical laboratory
Sample Imaging:	▪ Photography of selected sample section(s) ▪ Test method according to ISO 19901-8:2014
Geotechnical Index:	▪ Water content according to ISO 17892-1:2014 ▪ Unit weight derived from volume-mass calculation according to ISO 17892-2:2014 ▪ Unit weight derived from water content according to ISO 17892-2:2014

Office Geotechnical Laboratory	
General	
Test Programme:	▪ According to project specifications and within applicability of test type/method ▪ ISO standards are primary, where applicable
Programme Adjustment:	▪ Assessment of feasibility of a test (by sample inspection) prior to start of specimen preparation

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	▪ Decision by laboratory: (1) to proceed with test, (2) not to proceed with test, (3) to advise adjustments to test procedure ▪ Selection of alternative test specimen by laboratory and/or geotechnical engineer, if applicable in case of decision 'not to proceed' or when adequate test completion proves impracticable
Sample Batching:	Not applicable
Ground Description:	▪ According to document titled 'Soil Description and Classification' presented in the appendix titled 'Descriptions of Methods and Practices', as applicable ▪ Based on ISO 14688-1:2017, ISO 14688-2:2017, and BS 5930:2015+A1:2020 ▪ Geotechnical sample description is an interpretation of processed data available and/or selected at the time of preparation, where performed in office geotechnical laboratory
Sample Imaging:	▪ Photography of selected sample sections(s) ▪ Test method according to ISO 19901-8:2014
Geotechnical Index	
Water Content:	Not applicable
Atterberg Limits:	Not applicable
Unit Weight – Water Content:	Not applicable
Unit Weight – Volume-Mass:	Not applicable
Particle Density:	▪ According to ISO 17892-3:2015 and Fugro Test Method EUAF-FGBM-SOIL-TM-104 ▪ Small (fluid) pycnometer method ▪ Results show average value of two tests on one sample
Particle Size Distribution: – Sieving	According to ISO 17892-4:2016 and Fugro Test Method EUAF-FGBM-SOIL-TM-402
Particle Size Distribution: – Sedimentation	▪ According to ISO 17892-4:2016 and Fugro Test Method EUAF-FGBM-SOIL-TM-403 ▪ Pipette method ▪ On specimen for which sieving yielded percentage fines (fraction < 0.063 mm) > 15 %
Strength – Consolidated Drained Triaxial in Compression – Reconstituted Specimen	
Test Method:	▪ According to ISO 17892-9:2018 and Fugro Test Method EUAF-FGBM-SOIL-TM-701 ▪ Single test
Test Specimen:	▪ Coarse soil, cohesionless, reconstituted ▪ Target is no re-use of specimen material for consecutive tests

	▪ Laboratory-prepared, compacted to a target dry density, following the undercompaction technique (Ladd, 1978) and using a water content of ~10 % ▪ Target dry density is a best estimate derived from correlation with CPT results (relative density and soil behaviour type) ▪ Practical upper limit of specimen density corresponds to approximately 95 % relative density
Test Conditions:	No project-specific requirements
Specimen Saturation:	According to Test Method
Consolidation:	(An)isotropic consolidation to estimated in situ stress conditions
Shearing:	▪ According to Test Method ▪ Shearing to 20 % axial strain
Results:	According to Test Method
Interface Shear – Ring Shear – Soil-steel Interface	
Test Method:	▪ ICP method according to Jardine et al. (2005) and Fugro Test Method EUAF-FGBM-SOIL-TM-407 ▪ Residual shear strength measurement using Bromhead apparatus ▪ Two test stages with varying rates of angular displacement and normal effective stresses (stage 1: fast shear; stage 2: slow shear)
Steel Interface:	Steel roughness R_{cla} of 3 µm to 12.5 µm, defined as centreline average roughness according to ISO 8503-1:2012
Test Specimen - Coarse Soil:	▪ Coarse soil, cohesionless (with maximum particle size up to medium sand) ▪ Target is no re-use of specimen material for consecutive tests ▪ Test specimen is poured into the annulus of the ring shear apparatus in at least two layers, compacted by thumb pressure, and levelled across the upper surface using a suitably calibrated straight edge
Test Conditions:	▪ Stage 1: normal effective stress equivalent to estimated effective in situ horizontal stress; rate of linear displacement of 500 mm/min (fast shear) ▪ Stage 2: normal effective stress equivalent to 1.67 x estimated effective in situ horizontal stress; rate of linear displacement of 0.018 mm/min (slow shear) ▪ Estimated effective in situ vertical stress derived from soil unit weight profile

	▪ Estimated effective in situ horizontal stress derived from in situ vertical effective stress and selected value for coefficient of earth pressure at rest (K_0) ▪ Estimated K_0 from CPT-based correlations for overconsolidation ratio (e.g. Mayne, 2017) and the equation $K_0 = 0.45 \cdot \sqrt{OCR}$
Specimen Saturation:	No specific procedures for ensuring specimen saturation
Consolidation:	According to Test Method and as per Test Conditions
Shearing:	According to Test Method and as per Test Conditions
Results:	According to Test Method
Strength – Direct Shear	
Test Method:	▪ According to BS 1377-7:1990 ▪ Small shear box ▪ Square container (60 mm x 60 mm)
Test Specimen:	▪ Coarse soil, cohesionless, reconstituted ▪ Target is no re-use of specimen material for consecutive tests ▪ Laboratory-prepared, compacted to a target dry density ▪ Target dry density is a best estimate derived from correlation with CPT results (relative density and soil behaviour type)
Specimen Saturation:	No specific procedures for ensuring specimen saturation
Consolidation:	According to Test Method and as per Test Conditions
Shearing:	According to Test Method and as per Test Conditions
Results:	According to Test Method

D.1.3 References

- Computer Program GeODin®. *Recording, Presentation and Analysis of Geo-data.*
- BSI. (1990). *Methods of test for soils for civil engineering purposes - Shear strength tests (total stress)* (BS 1377-7:1990). British Standards Institution. https://knowledge.bsigroup.com/products/code-of-practice-for-ground-investigations/standard
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- ISO. (2012). *Preparation of steel substrates before application of paints and related products - surface roughness characteristics of blast-cleaned steel substrates - Part 1: Specifications and definitions for ISO surface profile comparators for the assessment of abrasive blast-cleaned surfaces* (ISO 8503-1:2012). International Organization for Standardization. https://www.iso.org/standard/51979.html

Contents Appendix D.2: Results of Classification Tests

List of Plates

Title	Plate
Laboratory Classification Test Results	D.2.1-1 to D.2.1-6
Particle Size Distribution Test Results	D.2.2-1 to D.2.2-10

LABORATORY CLASSIFICATION TEST RESULTS

LOCATION NEPL7-BH01

Sample		Test Depth	w	Unit Weight [kN/m³]				ρ_S	Carb. Cont.	Org. Cont.	Atterberg Limits			Fines	s_u [kPa]			
No.	Ground Description			γ_1	γ_2	γ_d min.	γ_d max.				w_P	w_L	I_P		PP	TV	FC	LV
		[m]	[%]					[Mg/m³]	[%]	[%]	[%]	[%]	[%]	[%]				
01	From 0.00 m to 0.45 m - light olive brown slightly silty fine and medium SAND with frequent fine to medium gravel-size shell fragments	0.00																
	from 0.00 - 0.20 m dark greyish brown																	
		0.20	23.3	19.8	19.7									4				
			23.3	19.8														
		0.45	20.5	20.3	20.8													
			20.5	20.3														
02	From 0.50 m to 0.60 m - dark grey slightly silty fine and medium calcareous SAND with frequent fine gravel-size shell fragments and with rare coarse sand size fragments of organic matter	0.50						2.69										
	From 0.60 m to 0.75 m - dark grey slightly silty fine and medium calcareous SAND with rare coarse sand-size fragments of organic matter	0.60	21.2	20.2	19.9													
		0.75	21.1	20.2	20.6													
03	From 1.00 m to 1.25 m - very dark grey slightly silty fine and medium calcareous SAND with rare coarse sand-size fragments of organic matter	1.00																
	From 1.25 m to 1.55 m - grey slightly silty fine and medium calcareous SAND with rare fine gravel-size shell fragments	1.25	22.2	20.0	19.4													
		1.55	22.7	19.9	20.7													
04	From 2.00 m to 2.60 m - grey slightly silty fine and medium calcareous SAND with rare fine gravel-size shell fragments	2.00																
		2.30	22.8	19.9	20.9									7				
		2.60	25.3	19.5	21.6													
05	From 31.00 m to 31.25 m - dark grey slightly silty fine and medium slightly calcareous SAND with rare coarse sand-size shell fragments and with rare fine sand-size mica crystals	31.00	28.7	19.5	18.2			2.67						8				
			25.6	19.0														

Key:
- w : water content
- γ_1 : unit weight derived from water content
- γ_2 : unit weight derived from volume mass calculation
- γ_{dmin} : minimum index unit weight
- γ_{dmax} : maximum index unit weight
- ρ_S : density of solid particles
- Carb.Cont. : carbonate content
- Org.Cont. : organic matter content
- w_P : plastic limit
- w_L : liquid limit
- I_P : plasticity index
- Fines : mass percentage of material passing 63 µm or 75 µm sieve
- PP : pocket penetrometer
- TV : torvane
- FC : fall cone
- LV : laboratory vane
- s_u : undrained shear strength
- 10r : r refers to test on remoulded soil
- 10d : d refers to test on disturbed soil

LABORATORY CLASSIFICATION TEST RESULTS

LOCATION NEPL7-BH01

Sample No.	Ground Description	Test Depth [m]	w [%]	Unit Weight [kN/m³] γ_1	γ_2	γ_d min.	γ_d max.	ρ_S [Mg/m³]	Carb. Cont. [%]	Org. Cont. [%]	Atterberg Limits w_P [%]	w_L [%]	I_P [%]	Fines [%]	s_u [kPa] PP	TV	FC	LV
	From 31.25 m to 31.80 m - very dark grey silty fine calcareous SAND with rare coarse sand-size shell fragments and with rare fine sand-size mica crystals	31.25																
		31.50	15.5	21.3										37				
		31.80	14.2 16.0	21.2 21.6	19.4													
06	From 34.50 m to 34.75 m - dark grey slightly silty fine and medium slightly calcareous SAND with frequent coarse sand-size shell fragments and with occasional mica crystals from 34.55 m to 34.70 m - extremely closely spaced thin to thick laminae of abundant coarse sand-size to fine gravel-size shell fragments and with thin laminae of soft grey non-calcareous clay and thin to thick laminae of dark brown possible organic matter at 34.60 m - thick lamina of wood fragments	34.50	25.9	19.4														
07	From 35.00 m to 35.05 m - dark grey slightly silty fine and medium slightly calcareous SAND with frequent coarse sand-size shell fragments and with occasional fine sand-size mica crystals and with occasional fine and medium sand-size possible organic matter	35.00	25.9	19.4														
08	From 38.00 m to 38.15 m - dark grey very clayey fine slightly calcareous SAND with occasional mica crystals	38.00	19.6 19.9	20.4 20.5	20.6													
	From 38.15 m to 38.30 m - dark grey slightly silty fine slightly calcareous SAND with occasional mica crystals	38.15												5				
		38.30	25.4 25.6	19.5 19.5	19.0													
09	From 38.50 m to 38.50 m - no recovery	38.50						2.67										
09A	From 38.50 m to 38.65 m - grey slightly silty fine slightly calcareous SAND with occasional mica crystals	38.50	24.0	19.7														
	From 38.65 m to 38.80 m - grey clayey fine slightly calcareous SAND with rare coarse sand-size to fine gravel-size shell fragments and with rare mica crystals	38.65																

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LABORATORY CLASSIFICATION TEST RESULTS

LOCATION NEPL7-BH01

Sample		Test Depth	w	Unit Weight [kN/m³]				ρ_S	Carb. Cont.	Org. Cont.	Atterberg Limits			Fines	s_u [kPa]			
No.	Ground Description			γ_1	γ_2	γ_d min.	γ_d max.				w_P	w_L	I_P		PP	TV	FC	LV
		[m]	[%]					[Mg/m³]	[%]	[%]	[%]	[%]	[%]	[%]				
		38.80	26.8	19.3	19.0													
			26.8	19.3														
10	From 42.50 m to 42.70 m - grey slightly silty fine and medium slightly calcareous SAND with rare mica crystals At 42.65 m - a medium gravel-size pocket of fine gravel-size wood fragments	42.50	23.7	19.8										5				
		42.70	23.9	19.7	19.3													
			23.9	19.7														
11	From 43.00 m to 43.15 m - grey slightly silty fine and medium slightly calcareous SAND with rare mica crystals From 43.10 m to 43.15 m - becoming slightly clayey	43.00	25.8	19.4														
		43.15	25.7	19.5	19.7													
			25.6	19.4														
12	From 46.50 m to 46.60 m - grey slightly silty fine and medium SAND slightly calcareous with rare mica crystals	46.50																
		46.60	27.7	19.2	19.4													
			27.3	19.1														
13	From 47.00 m to 47.20 m - grey fine and medium calcareous SAND with rare mica crystals	47.00	24.2	19.5	20.0													
			25.1	19.7														
		47.20	24.1	19.5	20.0													
			25.1	19.7														
14	From 50.50 m to 50.70 m - grey fine and medium calcareous SAND with rare mica crystals	50.50						2.67						5				
		50.70	23.8	19.6	18.1													
			24.6	19.7														
15	From 51.00 m to 51.10 m - grey fine and medium calcareous SAND with rare mica crystals	51.00																
		51.10	24.3	19.5	19.1													
			25.3	19.7														
16	From 54.50 m to 54.90 m - grey fine and medium calcareous SAND with rare coarse sand-size shell fragments with rare mica crystals	54.50																

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- I_P : plasticity index
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- FC : fall cone
- LV : laboratory vane
- s_u : undrained shear strength
- 10r : r refers to test on remoulded soil
- 10d : d refers to test on disturbed soil

LABORATORY CLASSIFICATION TEST RESULTS

LOCATION NEPL7-BH01

Sample		Test Depth	w	Unit Weight [kN/m³]				ρ_S	Carb. Cont.	Org. Cont.	Atterberg Limits			Fines	s_u [kPa]			
No.	Ground Description			γ_1	γ_2	γ_d min.	γ_d max.				w_P	w_L	I_P		PP	TV	FC	LV
		[m]	[%]					[Mg/m³]	[%]	[%]	[%]	[%]	[%]	[%]				
		54.70	24.1	19.7	19.1													
		54.90	22.8	19.9														
17	From 58.00 m to 58.25 m - grey slightly clayey fine and medium calcareous SAND with rare mica crystals	58.00																
	From 58.25 m to 58.75 m - grey slightly clayey fine and medium calcareous SAND with extremely closely spaced thin laminae of black organic matter and with rare mica crystals	58.25	26.5	19.3	19.9													
		58.50	27.4	19.2	18.7													
		58.75	25.9	19.4														
18	From 61.50 m to 62.10 m - grey fine and medium calcareous SAND with rare mica crystals	61.50																
		61.80	23.9	19.7	19.3									17				
		61.95	23.4	19.8	18.8													
		62.10	19.0	20.6														
19	From 65.00 m to 65.15 m - grey fine and medium calcareous SAND with occasional fine gravel-size pockets of grey clay and with rare mica crystals	65.00																
	From 65.15 m to 65.30 m - grey fine and medium calcareous SAND with rare mica crystals	65.15	18.5	20.7														
		65.30	24.1	19.7	19.1													
20	From 68.00 m to 68.15 m - grey fine and medium calcareous SAND with rare mica crystals	68.00												7				
		68.05	24.8	19.6														
		68.15	25.6	19.5	19.0													
21	From 71.50 m to 71.60 m - grey fine and medium calcareous SAND with rare mica crystals	71.50																
		71.55	22.0	20.0														

Key:
- w : water content
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- γ_2 : unit weight derived from volume mass calculation
- γ_{dmin} : minimum index unit weight
- γ_{dmax} : maximum index unit weight
- ρ_S : density of solid particles
- Carb.Cont. : carbonate content
- Org.Cont. : organic matter content
- w_P : plastic limit
- w_L : liquid limit
- I_P : plasticity index
- Fines : mass percentage of material passing 63 µm or 75 µm sieve
- PP : pocket penetrometer
- TV : torvane
- FC : fall cone
- LV : laboratory vane
- s_u : undrained shear strength
- 10r : r refers to test on remoulded soil
- 10d : d refers to test on disturbed soil

LABORATORY CLASSIFICATION TEST RESULTS

LOCATION NEPL7-BH01

Sample		Test Depth	w	Unit Weight [kN/m³]				ρ_S	Carb. Cont.	Org. Cont.	Atterberg Limits			Fines	s_u [kPa]			
No.	Ground Description			γ_1	γ_2	γ_d min.	γ_d max.				w_P	w_L	I_P		PP	TV	FC	LV
		[m]	[%]					[Mg/m³]	[%]	[%]	[%]	[%]	[%]	[%]				
		71.60	23.9	19.7	19.3													
22	From 75.00 m to 75.15 m - grey fine and medium calcareous SAND with rare mica crystals	75.00																
		75.05	24.7	19.6														
		75.15	26.4	19.3	18.4													
23	From 78.50 m to 78.65 m - dark grey slightly clayey fine and medium calcareous SAND with extremely closely spaced thin laminae of dark grey clay and with extremely closely spaced thin laminae of medium and coarse light grey sand	78.50	19.7	20.4										26				
24	From 82.00 m to 82.10 m - grey fine and medium SAND with rare mica crystals	82.00	24.1	19.7	19.1													
		82.05	25.2	19.5														
25	From 86.50 m to 86.60 m - grey fine and medium SAND with rare mica crystals	86.50																
		86.55	24.2	19.7														
		86.60	24.3	19.6	18.2													
26	From 90.00 m to 90.10 m - grey fine and medium SAND with rare mica crystals	90.00																
		90.05	24.1	19.7														
27	From 93.50 m to 93.60 m - grey fine and medium SAND with rare mica crystals	93.50																
		93.60	21.7 20.1	20.4 20.1	20.5													
28	From 96.50 m to 96.60 m - grey fine and medium SAND with rare mica crystals	96.50																
		96.60	21.5 22.5	20.0 20.1	21.0													

Key:
- w : water content
- γ_1 : unit weight derived from water content
- γ_2 : unit weight derived from volume mass calculation
- γ_{dmin} : minimum index unit weight
- γ_{dmax} : maximum index unit weight
- ρ_S : density of solid particles
- Carb.Cont. : carbonate content
- Org.Cont. : organic matter content
- w_P : plastic limit
- w_L : liquid limit
- I_P : plasticity index
- Fines : mass percentage of material passing 63 µm or 75 µm sieve
- PP : pocket penetrometer
- TV : torvane
- FC : fall cone
- LV : laboratory vane
- s_u : undrained shear strength
- 10r : r refers to test on remoulded soil
- 10d : d refers to test on disturbed soil

LABORATORY CLASSIFICATION TEST RESULTS

LOCATION NEPL7-BH01

Sample		Test Depth	w	Unit Weight [kN/m³]				ρ_S	Carb. Cont.	Org. Cont.	Atterberg Limits			Fines	s_u [kPa]			
No.	Ground Description			γ_1	γ_2	γ_d min.	γ_d max.				w_P	w_L	I_P		PP	TV	FC	LV
		[m]	[%]					[Mg/m³]	[%]	[%]	[%]	[%]	[%]	[%]				
29	From 99.50 m to 99.70 m - grey slightly silty fine SAND with rare mica crystals and rare medium sand-size to fine gravel-size possible organic matter including wood fragments	99.50																
		99.70	26.2 25.5	19.5 19.4	21.2													

Key:
- w : water content
- γ_1 : unit weight derived from water content
- γ_2 : unit weight derived from volume mass calculation
- γ_{dmin} : minimum index unit weight
- γ_{dmax} : maximum index unit weight
- ρ_S : density of solid particles
- Carb.Cont. : carbonate content
- Org.Cont. : organic matter content
- w_P : plastic limit
- w_L : liquid limit
- I_P : plasticity index
- Fines : mass percentage of material passing 63 µm or 75 µm sieve
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- LV : laboratory vane
- s_u : undrained shear strength
- 10r : r refers to test on remoulded soil
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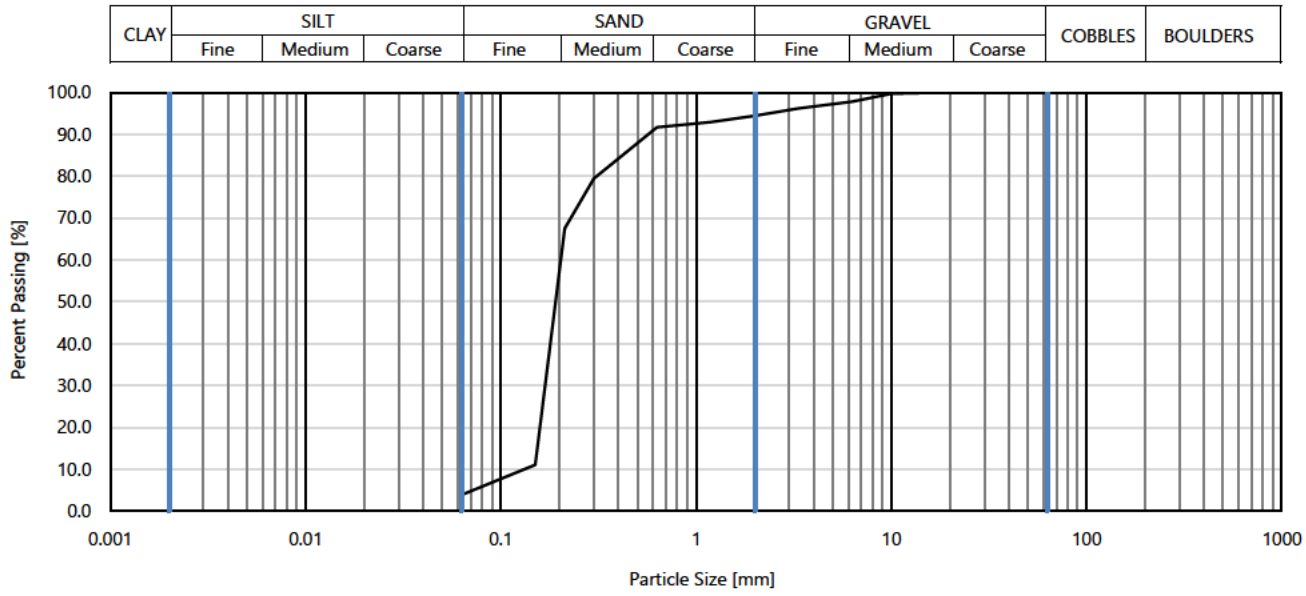
LABORATORY TEST CERTIFICATE

Determination of Particle Size Distribution

BS EN ISO 17892-4:2016 Clause 5.2

UKAS TESTING 1483

Project Reference	F225236			Location ID	NEPL7-BH01
Project Name	Neptune L7 -17			Depth Top [m]	0.20
Specimen Description	Brown slightly silty gravelly SAND			Sample Type	B
Specimen Reference		Specimen Depth [m]		Sample Reference	01BAGB



Sieving		Sedimentation	
Particle Size [mm]	Passing [%]	Particle Size [mm]	Passing [%]
125	100		
90.0	100		
75.0	100		
63.0	100		
37.5	100		
20.0	100		
10.0	100		
6.30	98		
3.35	96		
2.00	94		
1.18	93		
0.630	92		
0.300	79		
0.212	68		
0.150	11		
0.0630	4		

Dry Mass of Sample [g]	595
Particle Density [Mg/m³]	

Sample Proportions	Dry Mass [%]
Very coarse	0.0
Gravel	5.6
Sand	90.5
Fines <0.063mm	3.9

Grading Analysis	
D100 [mm]	20
D60 [mm]	0.202
D30 [mm]	0.168
D10 [mm]	0.131
Coefficient of Uniformity	1.5
Coefficient of Curvature	1.1

Issue Date	04/05/2023	Certificate Reference		Authorised by	tomlinsona
Client	Neptune			Authorised Date	27/04/2023
Remarks:					

Fugro GeoServices Ltd. Unit 43, Number One Industrial Estate, Medomsley Road, Consett, DH8 6TW

Testing was performed at the Fugro GeoServices Ltd laboratory at the address shown above. Results relate only to the sample tested, having been authorised by persons qualified to do so. Opinions and interpretations are outside the scope of accreditation. Unless stated otherwise the sample was tested in the condition it was received at the laboratory.

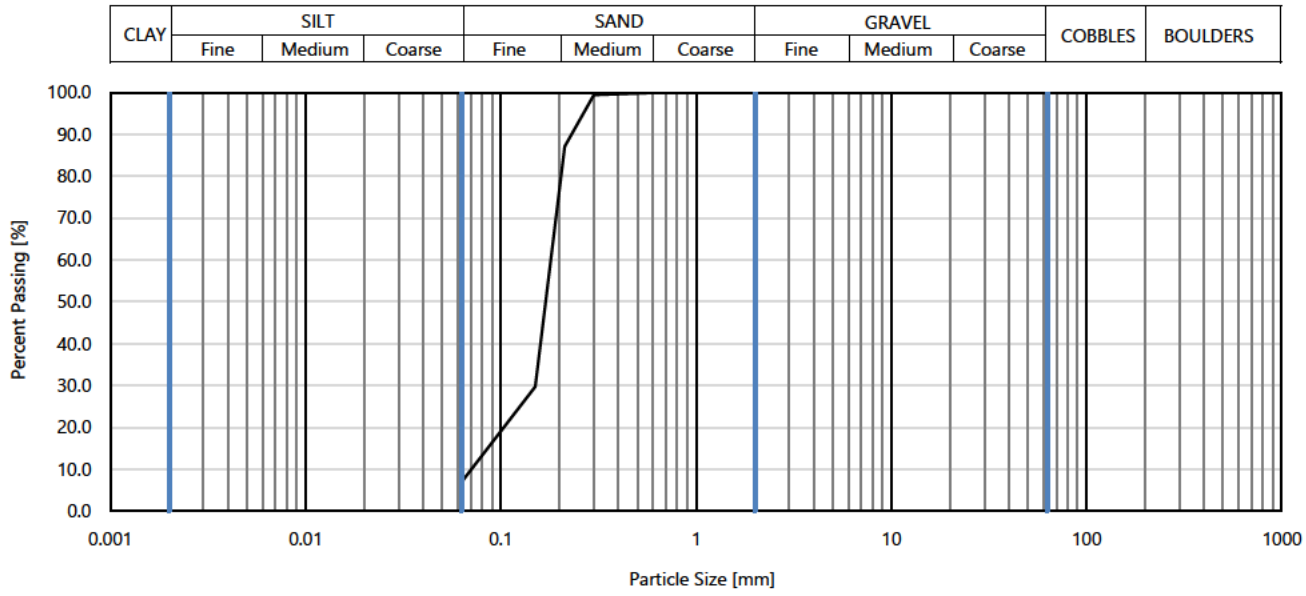
LABORATORY TEST CERTIFICATE

Determination of Particle Size Distribution

BS EN ISO 17892-4:2016 Clause 5.2

UKAS TESTING 1483

Project Reference	F225236			Location ID	NEPL7-BH01
Project Name	Neptune L7 -17			Depth Top [m]	2.30
Specimen Description	Brown slightly gravelly silty SAND			Sample Type	B
Specimen Reference		Specimen Depth [m]		Sample Reference	04BAGB



Sieving		Sedimentation	
Particle Size [mm]	Passing [%]	Particle Size [mm]	Passing [%]
125	100		
90.0	100		
75.0	100		
63.0	100		
37.5	100		
20.0	100		
10.0	100		
6.30	100		
3.35	100		
2.00	100		
1.18	100		
0.630	100		
0.300	99		
0.212	87		
0.150	30		
0.0630	7		

Dry Mass of Sample [g]	1130
Particle Density [Mg/m³]	

Sample Proportions	Dry Mass [%]
Very coarse	0.0
Gravel	0.1
Sand	92.9
Fines <0.063mm	7.0

Grading Analysis	
D100 [mm]	3.35
D60 [mm]	0.18
D30 [mm]	0.15
D10 [mm]	0.0706
Coefficient of Uniformity	2.5
Coefficient of Curvature	1.8

Issue Date	04/05/2023	Certificate Reference		Authorised by	tomlinsona
Client	Neptune			Authorised Date	27/04/2023
Remarks:					

Fugro GeoServices Ltd. Unit 43, Number One Industrial Estate, Medomsley Road, Consett, DH8 6TW

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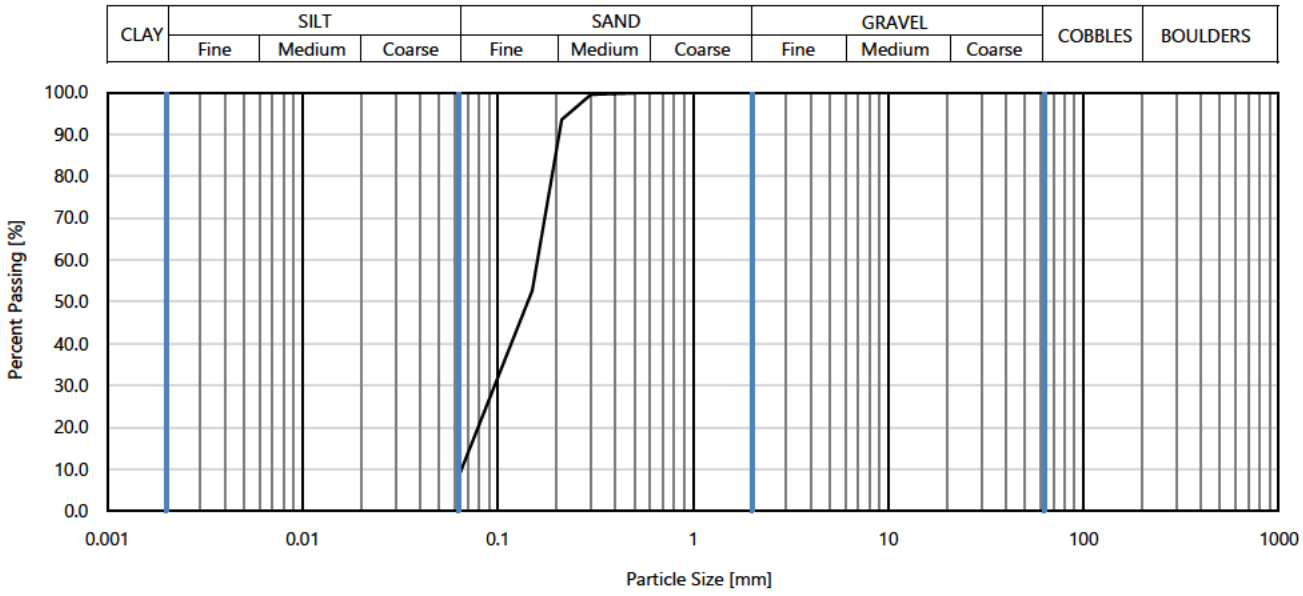
LABORATORY TEST CERTIFICATE

Determination of Particle Size Distribution

BS EN ISO 17892-4:2016 Clause 5.2

UKAS TESTING 1483

Project Reference	F225236			Location ID	NEPL7-BH01
Project Name	Neptune L7 -17			Depth Top [m]	31.00
Specimen Description	Grey silty SAND			Sample Type	B
Specimen Reference		Specimen Depth [m]		Sample Reference	05BAGA



Sieving		Sedimentation	
Particle Size [mm]	Passing [%]	Particle Size [mm]	Passing [%]
125	100		
90.0	100		
75.0	100		
63.0	100		
37.5	100		
20.0	100		
10.0	100		
6.30	100		
3.35	100		
2.00	100		
1.18	100		
0.630	100		
0.300	100		
0.212	94		
0.150	53		
0.0630	8		

Dry Mass of Sample [g]	633
Particle Density [Mg/m³]	

Sample Proportions	Dry Mass [%]
Very coarse	0.0
Gravel	0.0
Sand	91.7
Fines <0.063mm	8.3

Grading Analysis	
D100 [mm]	2
D60 [mm]	0.16
D30 [mm]	0.0963
D10 [mm]	0.0651
Coefficient of Uniformity	2.5
Coefficient of Curvature	0.89

Issue Date	04/05/2023	Certificate Reference		Authorised by	tomlinsona
Client	Neptune			Authorised Date	27/04/2023
Remarks:					

Fugro GeoServices Ltd. Unit 43, Number One Industrial Estate, Medomsley Road, Consett, DH8 6TW

Testing was performed at the Fugro GeoServices Ltd laboratory at the address shown above. Results relate only to the sample tested, having been authorised by persons qualified to do so. Opinions and interpretations are outside the scope of accreditation. Unless stated otherwise the sample was tested in the condition it was received at the laboratory.

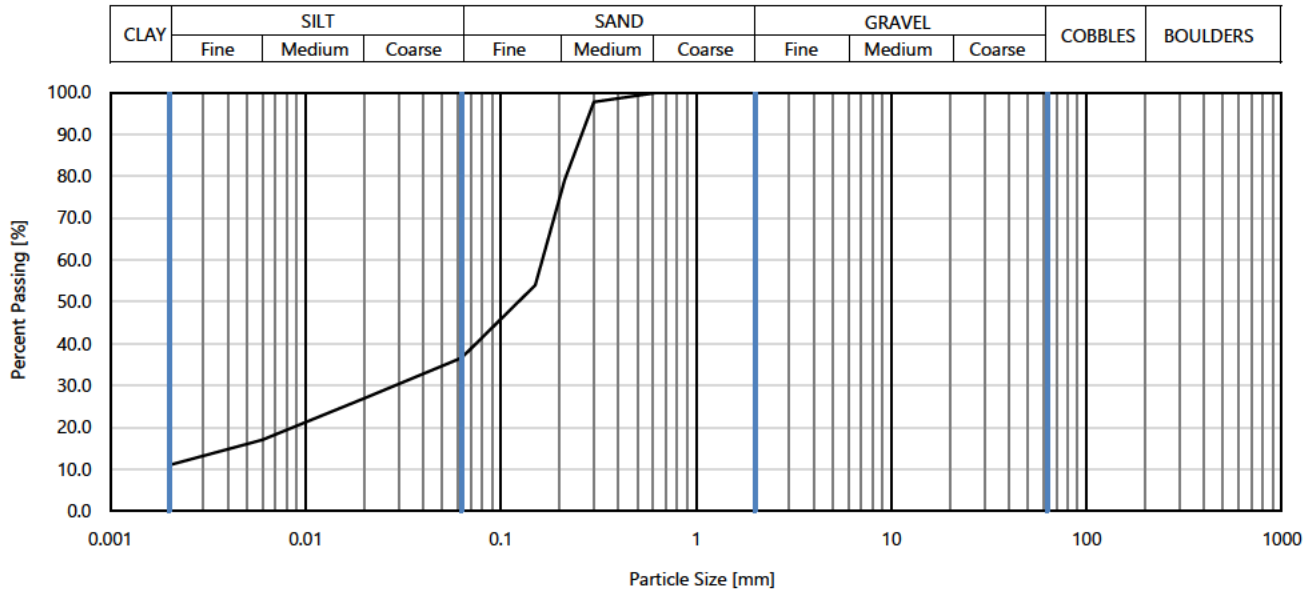
LABORATORY TEST CERTIFICATE

Determination of Particle Size Distribution

BS EN ISO 17892-4:2016 Clauses 5.2 and 5.4

UKAS TESTING 1483

Project Reference	F225236			Location ID	NEPL7-BH01
Project Name	Neptune L7 -17			Depth Top [m]	31.50
Specimen Description	Grey sandy CLAY			Sample Type	B
Specimen Reference		Specimen Depth [m]		Sample Reference	05BAGC



Sieving		Sedimentation	
Particle Size [mm]	Passing [%]	Particle Size [mm]	Passing [%]
125	100	0.0200	27
90.0	100	0.00600	17
75.0	100	0.00200	11
63.0	100		
37.5	100		
20.0	100		
10.0	100		
6.30	100		
3.35	100		
2.00	100		
1.18	100		
0.630	100		
0.300	98		
0.212	79		
0.150	54		
0.0630	37		

Dry Mass of Sample [g]	911
Particle Density [Mg/m³]	2.70 assumed

Sample Proportions	Dry Mass [%]
Very coarse	0.0
Gravel	0.0
Sand	63.4
Silt	25.5
Clay	11.1

Grading Analysis	
D100 [mm]	3.35
D60 [mm]	0.163
D30 [mm]	0.0281
D10 [mm]	-
Coefficient of Uniformity	Not applicable
Coefficient of Curvature	Not applicable

Issue Date	04/05/2023	Certificate Reference		Authorised by	tomlinsona
Client	Neptune			Authorised Date	28/04/2023
Remarks:					

Fugro GeoServices Ltd. Unit 43, Number One Industrial Estate, Medomsley Road, Consett, DH8 6TW

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Checked by: LB Date: 11/05/2023
Approved by: PH Date: 12/05/2023

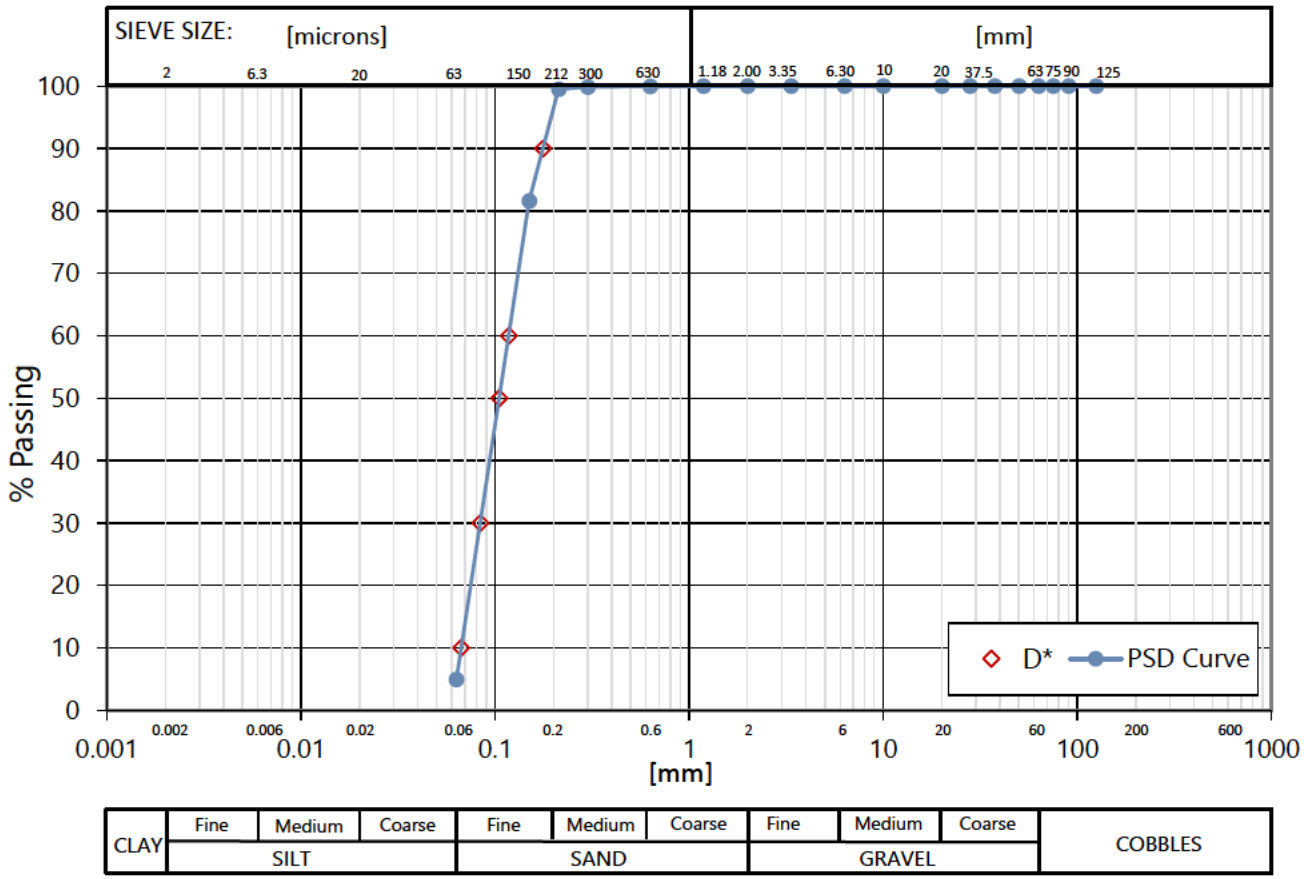
Template Issue: 5.0
Filename: 230238-F225236 / Labdata / Class / PSD / NEPL7-BH01_08BAGB_ps.XLS

Drawn by: IBG
Date: 26/04/2023

% Passing	Particle Size (mm)	
100	125	Sieving
100	90.0	
100	75.0	
100	63.0	
100	50.0	
100	37.5	
100	28.0	
100	20.0	
100	10.0	
100	6.30	
100	3.35	
100	2.00	
100	1.18	
100	0.630	
100	0.300	
100	0.212	
82	0.150	
5	0.0630	
		Sedimentation

Sieving:	
Initial dry mass of sample - sieve [g]	229.06
Sedimentation:	
Mass retained on 63µm sieve [g]	n/a

Grading Analysis		
D_{90}	0.1764	[mm]
D_{60}	0.1175	[mm]
D_{50}	0.1049	[mm]
D_{30}	0.0837	[mm]
D_{10}	0.0667	[mm]



Borehole	Sample	Depth [m]	Percentage soil types				
			Clay	Silt **	Sand	Gravel	Cobbles
NEPL7-BH01	08BAGB	38.15	0	5	95	0	0

** Note: Where a sedimentation test was not carried out, this figure represents total fines, i.e., particles of diameter less than 63 microns

Description	Remarks
Dark greyish brown silty fine SAND	

Tested in accordance with ISO 17892-4: 2016 - 5.2 - Sieving method

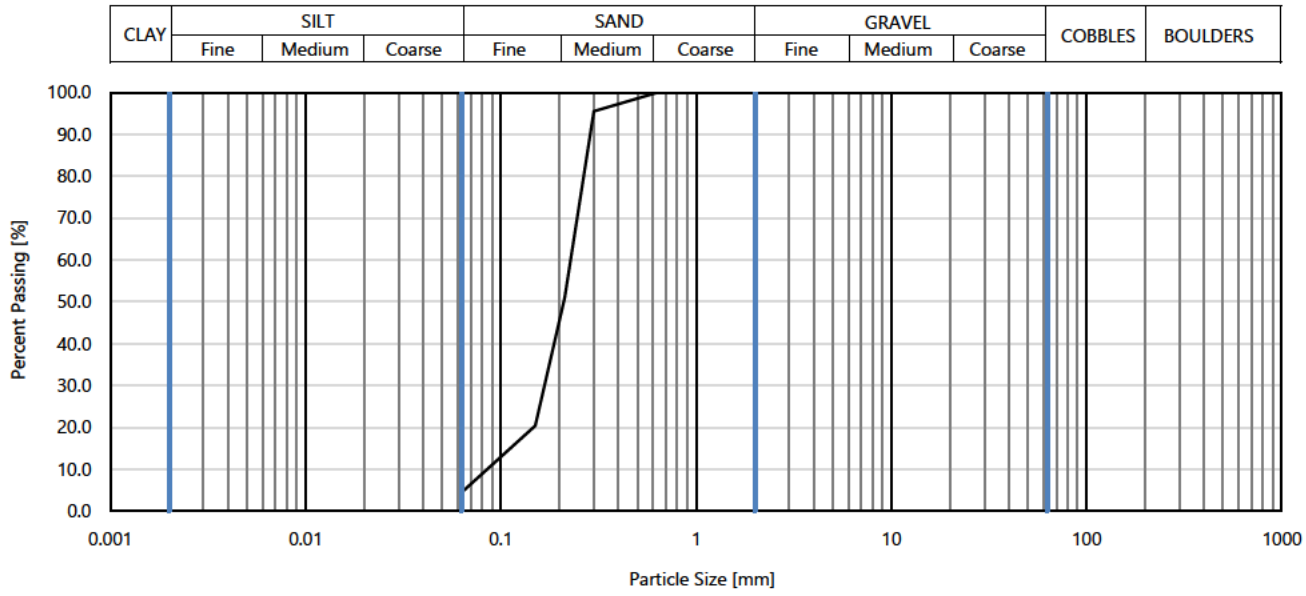
LABORATORY TEST CERTIFICATE

Determination of Particle Size Distribution

BS EN ISO 17892-4:2016 Clause 5.2

UKAS TESTING 1483

Project Reference	F225236			Location ID	NEPL7-BH01
Project Name	Neptune L7 -17			Depth Top [m]	42.50
Specimen Description	Brown slightly silty SAND			Sample Type	B
Specimen Reference		Specimen Depth [m]		Sample Reference	10BAGA



Sieving		Sedimentation	
Particle Size [mm]	Passing [%]	Particle Size [mm]	Passing [%]
125	100		
90.0	100		
75.0	100		
63.0	100		
37.5	100		
20.0	100		
10.0	100		
6.30	100		
3.35	100		
2.00	100		
1.18	100		
0.630	100		
0.300	96		
0.212	51		
0.150	20		
0.0630	5		

Dry Mass of Sample [g]	305
Particle Density [Mg/m³]	

Sample Proportions	Dry Mass [%]
Very coarse	0.0
Gravel	0.0
Sand	95.4
Fines <0.063mm	4.5

Grading Analysis	
D100 [mm]	3.35
D60 [mm]	0.227
D30 [mm]	0.167
D10 [mm]	0.0849
Coefficient of Uniformity	2.7
Coefficient of Curvature	1.4

Issue Date	04/05/2023	Certificate Reference		Authorised by	tomlinsona
Client	Neptune			Authorised Date	28/04/2023
Remarks:					

Fugro GeoServices Ltd. Unit 43, Number One Industrial Estate, Medomsley Road, Consett, DH8 6TW

Testing was performed at the Fugro GeoServices Ltd laboratory at the address shown above. Results relate only to the sample tested, having been authorised by persons qualified to do so. Opinions and interpretations are outside the scope of accreditation. Unless stated otherwise the sample was tested in the condition it was received at the laboratory.

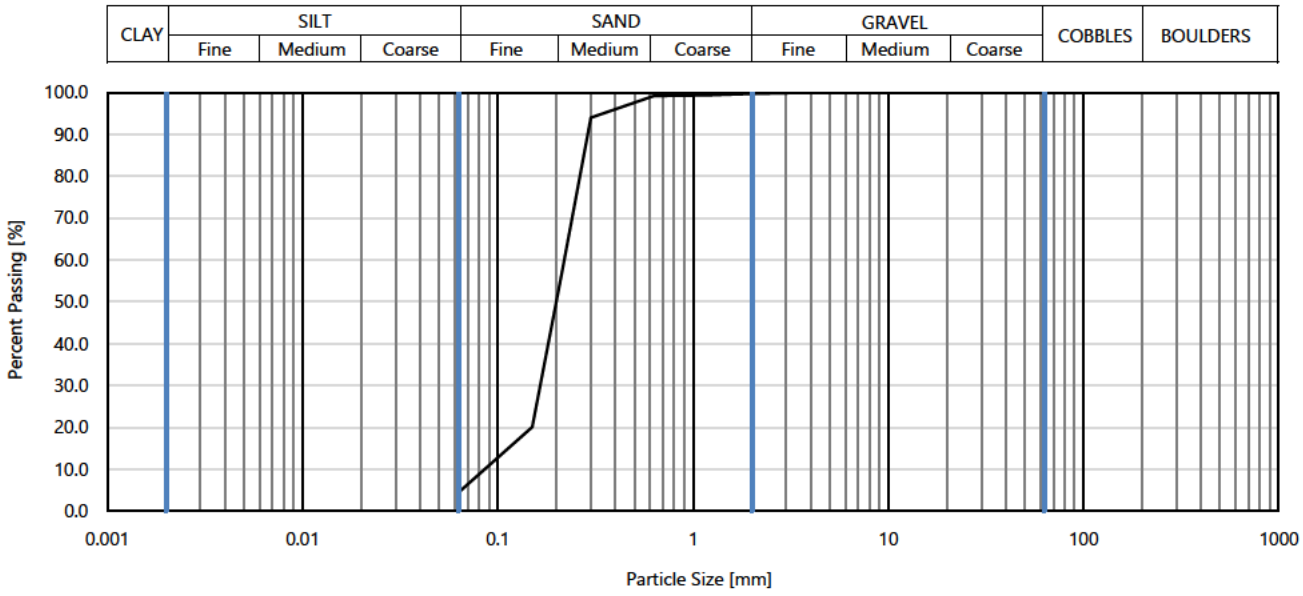
LABORATORY TEST CERTIFICATE

Determination of Particle Size Distribution

BS EN ISO 17892-4:2016 Clause 5.2

1483

Project Reference	F225236			Location ID	NEPL7-BH01
Project Name	Neptune L7 -17			Depth Top [m]	50.50
Specimen Description	Grey slightly gravelly slightly silty SAND			Sample Type	B
Specimen Reference		Specimen Depth [m]		Sample Reference	14BAGA



Sieving		Sedimentation	
Particle Size [mm]	Passing [%]	Particle Size [mm]	Passing [%]
125	100		
90.0	100		
75.0	100		
63.0	100		
37.5	100		
20.0	100		
10.0	100		
6.30	100		
3.35	100		
2.00	100		
1.18	99		
0.630	99		
0.300	94		
0.212	57		
0.150	20		
0.0630	5		

Dry Mass of Sample [g]	412
Particle Density [Mg/m³]	

Sample Proportions	Dry Mass [%]
Very coarse	0.0
Gravel	0.4
Sand	95.0
Fines <0.063mm	4.5

Grading Analysis	
D100 [mm]	6.3
D60 [mm]	0.219
D30 [mm]	0.165
D10 [mm]	0.0855
Coefficient of Uniformity	2.6
Coefficient of Curvature	1.5

Issue Date	04/05/2023	Certificate Reference		Authorised by	tomlinsona
Client	Neptune			Authorised Date	27/04/2023
Remarks:					

Fugro GeoServices Ltd. Unit 43, Number One Industrial Estate, Medomsley Road, Consett, DH8 6TW

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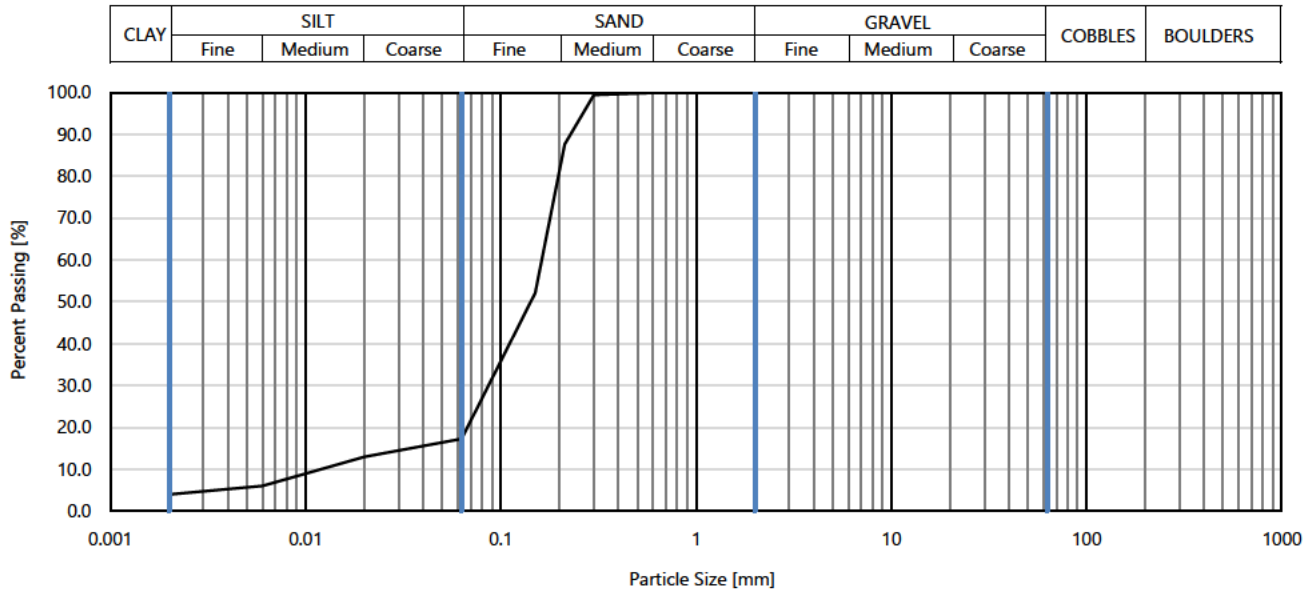
LABORATORY TEST CERTIFICATE

Determination of Particle Size Distribution

BS EN ISO 17892-4:2016 Clauses 5.2 and 5.4

UKAS TESTING 1483

Project Reference	F225236			Location ID	NEPL7-BH01
Project Name	Neptune L7 -17			Depth Top [m]	61.80
Specimen Description	Grey silty SAND			Sample Type	B
Specimen Reference		Specimen Depth [m]		Sample Reference	18BAGB



Sieving		Sedimentation	
Particle Size [mm]	Passing [%]	Particle Size [mm]	Passing [%]
125	100	0.0200	13
90.0	100	0.00600	6
75.0	100	0.00200	4
63.0	100		
37.5	100		
20.0	100		
10.0	100		
6.30	100		
3.35	100		
2.00	100		
1.18	100		
0.630	100		
0.300	99		
0.212	88		
0.150	52		
0.0630	17		

Dry Mass of Sample [g]	758
Particle Density [Mg/m³]	2.70 assumed

Sample Proportions	Dry Mass [%]
Very coarse	0.0
Gravel	0.0
Sand	82.7
Silt	13.2
Clay	4.1

Grading Analysis	
D100 [mm]	3.35
D60 [mm]	0.162
D30 [mm]	0.0865
D10 [mm]	0.0122
Coefficient of Uniformity	13
Coefficient of Curvature	3.8

Issue Date	04/05/2023	Certificate Reference		Authorised by	tomlinsona
Client	Neptune			Authorised Date	28/04/2023
Remarks:					

Fugro GeoServices Ltd. Unit 43, Number One Industrial Estate, Medomsley Road, Consett, DH8 6TW

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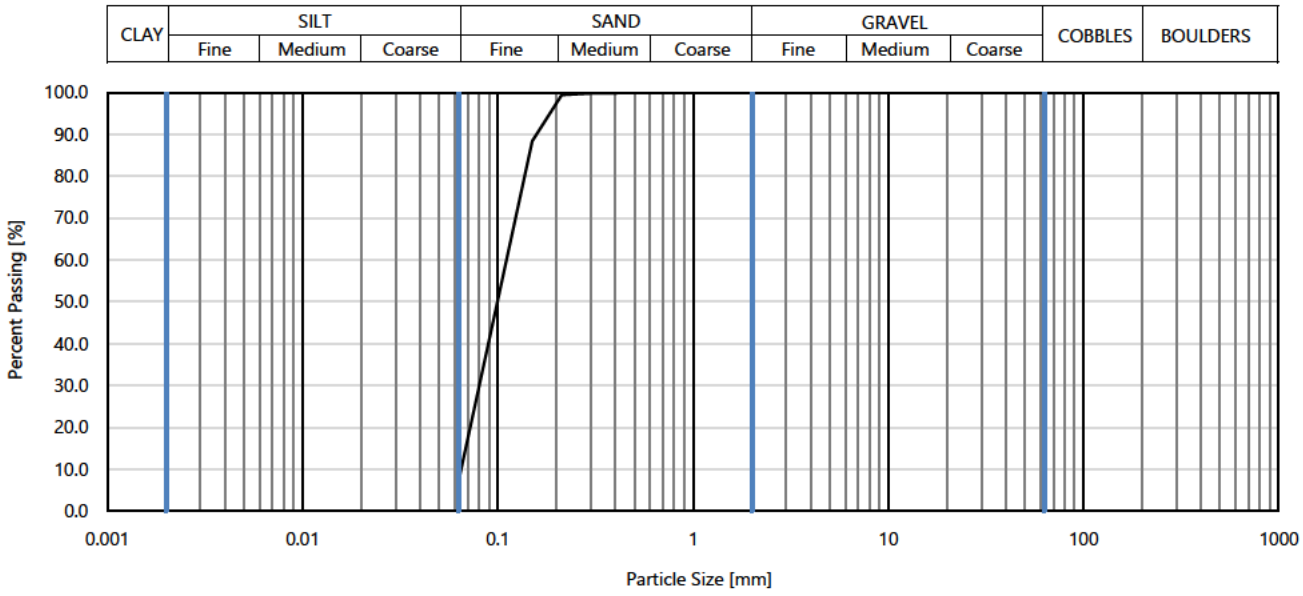
LABORATORY TEST CERTIFICATE

Determination of Particle Size Distribution

BS EN ISO 17892-4:2016 Clause 5.2

1483

Project Reference	F225236			Location ID	NEPL7-BH01
Project Name	Neptune L7 -17			Depth Top [m]	68.00
Specimen Description	Brown silty SAND			Sample Type	B
Specimen Reference		Specimen Depth [m]		Sample Reference	20BAGA



Sieving		Sedimentation	
Particle Size [mm]	Passing [%]	Particle Size [mm]	Passing [%]
125	100		
90.0	100		
75.0	100		
63.0	100		
37.5	100		
20.0	100		
10.0	100		
6.30	100		
3.35	100		
2.00	100		
1.18	100		
0.630	100		
0.300	100		
0.212	99		
0.150	88		
0.0630	7		

Dry Mass of Sample [g]	315
Particle Density [Mg/m³]	

Sample Proportions	Dry Mass [%]
Very coarse	0.0
Gravel	0.0
Sand	92.7
Fines <0.063mm	7.3

Grading Analysis	
D100 [mm]	2
D60 [mm]	0.111
D30 [mm]	0.0803
D10 [mm]	0.0648
Coefficient of Uniformity	1.7
Coefficient of Curvature	0.9

Issue Date	04/05/2023	Certificate Reference		Authorised by	tomlinsona
Client	Neptune			Authorised Date	27/04/2023
Remarks:					

Fugro GeoServices Ltd. Unit 43, Number One Industrial Estate, Medomsley Road, Consett, DH8 6TW

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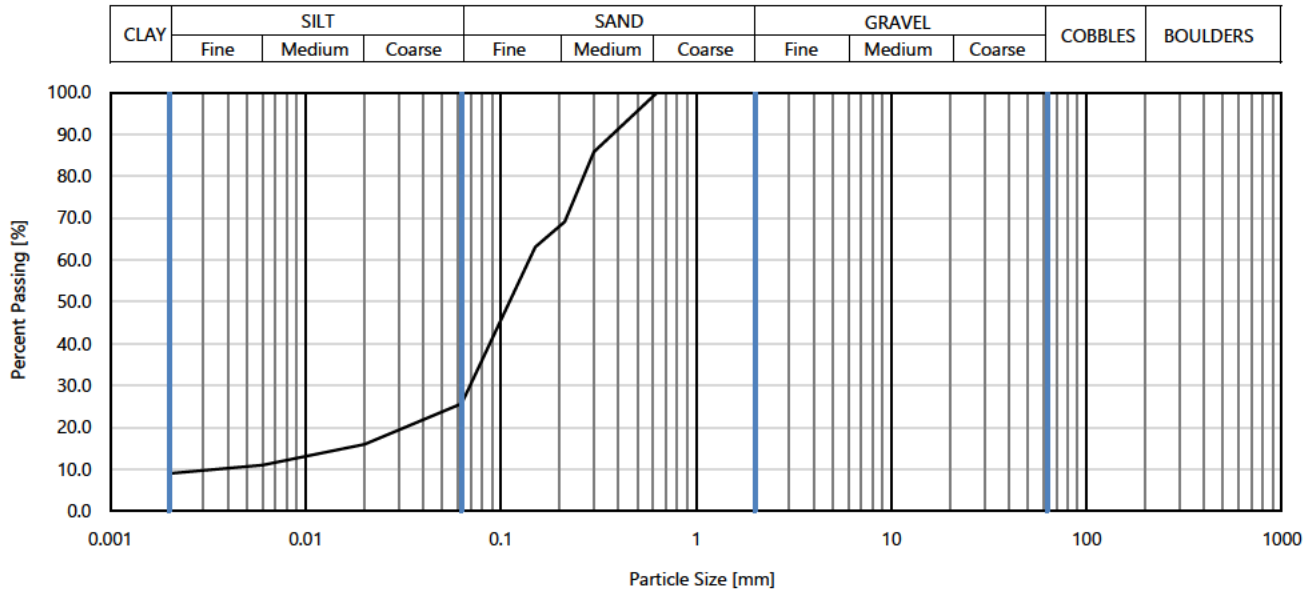
LABORATORY TEST CERTIFICATE

Determination of Particle Size Distribution

BS EN ISO 17892-4:2016 Clauses 5.2 and 5.4

UKAS TESTING 1483

Project Reference	F225236			Location ID	NEPL7-BH01
Project Name	Neptune L7 -17			Depth Top [m]	78.50
Specimen Description	Brown very clayey SAND			Sample Type	B
Specimen Reference		Specimen Depth [m]		Sample Reference	23BAGA



Sieving		Sedimentation	
Particle Size [mm]	Passing [%]	Particle Size [mm]	Passing [%]
125	100	0.0200	16
90.0	100	0.00600	11
75.0	100	0.00200	9
63.0	100		
37.5	100		
20.0	100		
10.0	100		
6.30	100		
3.35	100		
2.00	100		
1.18	100		
0.630	100		
0.300	86		
0.212	69		
0.150	63		
0.0630	26		

Dry Mass of Sample [g]	319
Particle Density [Mg/m³]	2.70 assumed

Sample Proportions	Dry Mass [%]
Very coarse	0.0
Gravel	0.0
Sand	74.3
Silt	16.9
Clay	8.8

Grading Analysis	
D100 [mm]	2
D60 [mm]	0.14
D30 [mm]	0.0696
D10 [mm]	0.00347
Coefficient of Uniformity	40
Coefficient of Curvature	10

Issue Date	04/05/2023	Certificate Reference		Authorised by	tomlinsona
Client	Neptune			Authorised Date	28/04/2023
Remarks:					

Fugro GeoServices Ltd. Unit 43, Number One Industrial Estate, Medomsley Road, Consett, DH8 6TW

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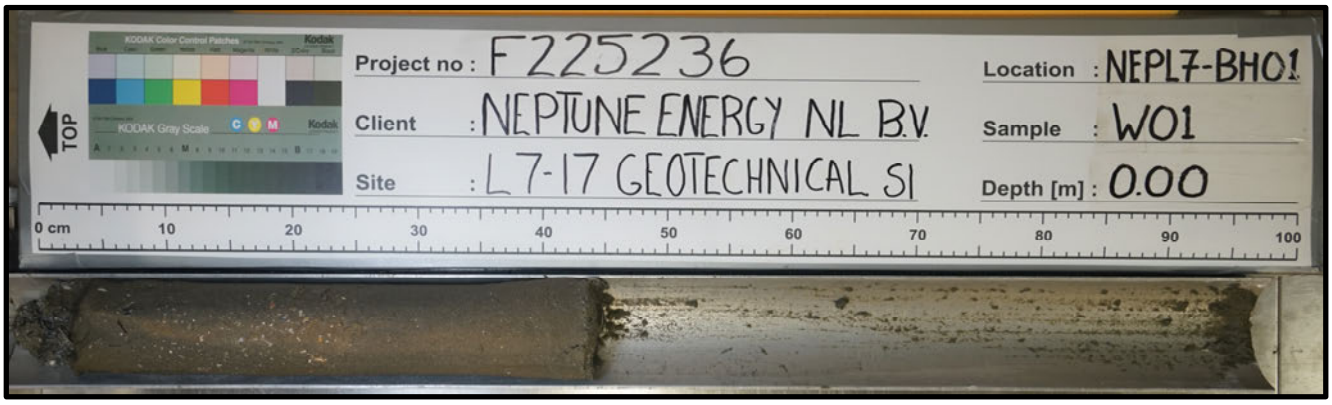
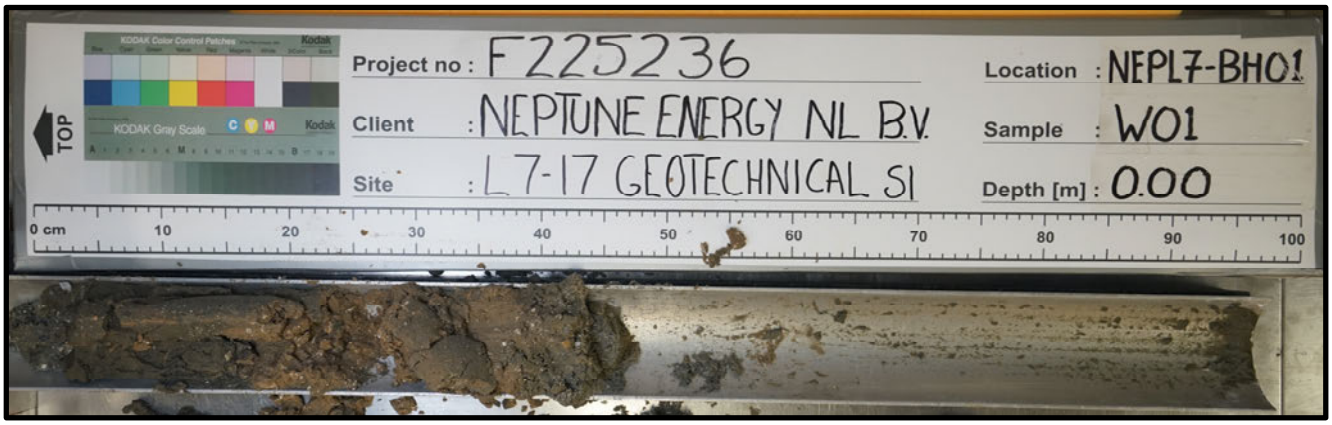
Contents Appendix D.3: Sample Photographs

List of Plates

Title	Plate
Sample Photographs	D.3-1 to D.3-30



Location	:	NEPL7-BH01
Sample	:	W01
Depth [m BSF]	:	0.00
Note(s)	:	Intact

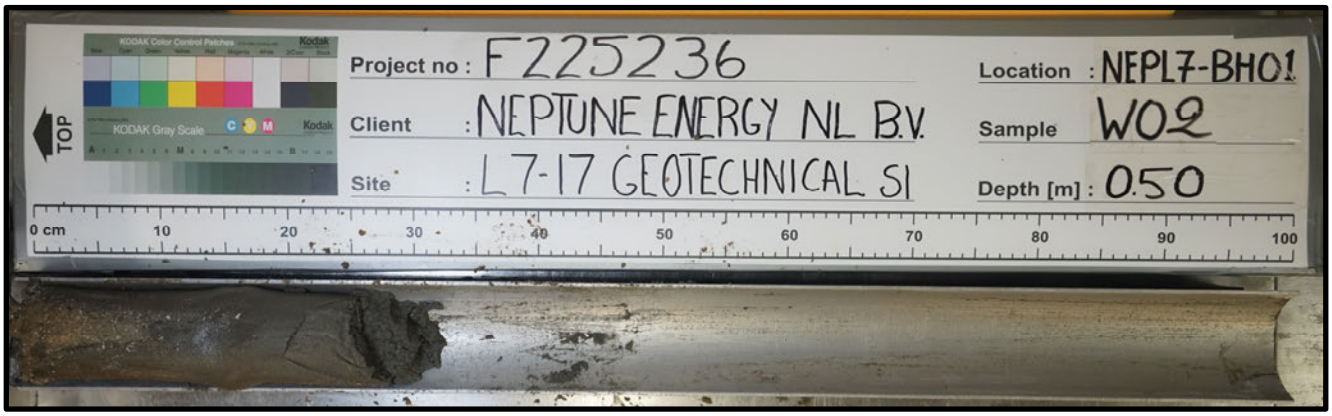


Location	:	NEPL7-BH01
Sample	:	W01
Depth [m BSF]	:	0.00
Note(s)	:	Split

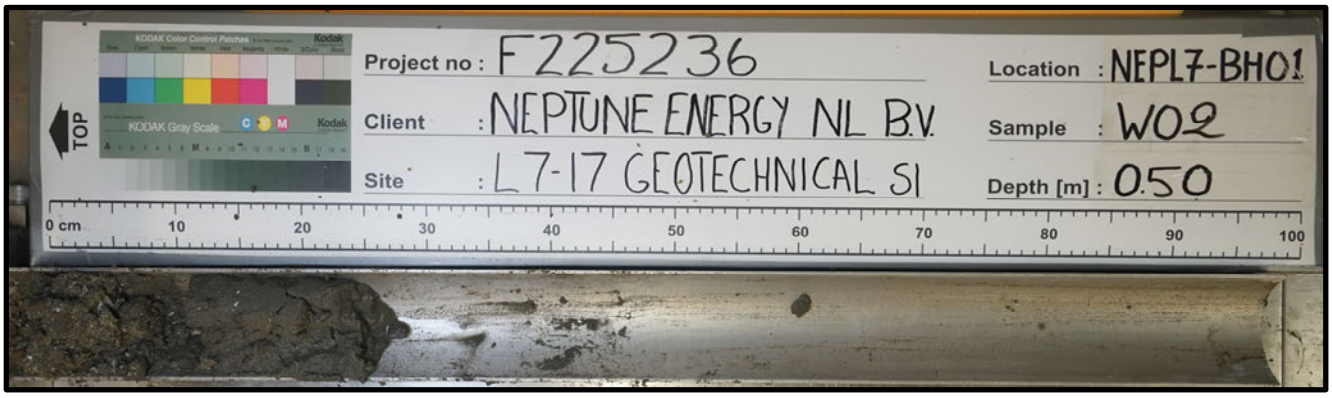
SAMPLE PHOTOGRAPHS

Release Date: 6-3-2023
Drawn with: Photo_Plate_Maker v 6.0
Photo Folder: I Project Workfolders\Neptune L7\Photos\Proc
Presented Plate: Downhole_Portrait
Processed by: DBS
Date: 6-3-2023

Release Date: 6-3-2023
Drawn with: Photo_Plate_Maker v 6.0
Photo Folder: I Project Workfolders\Neptune L7\Photos\Proc
Presented Plate: Downhole_Portrait
Processed by: DBS
Date: 6-3-2023



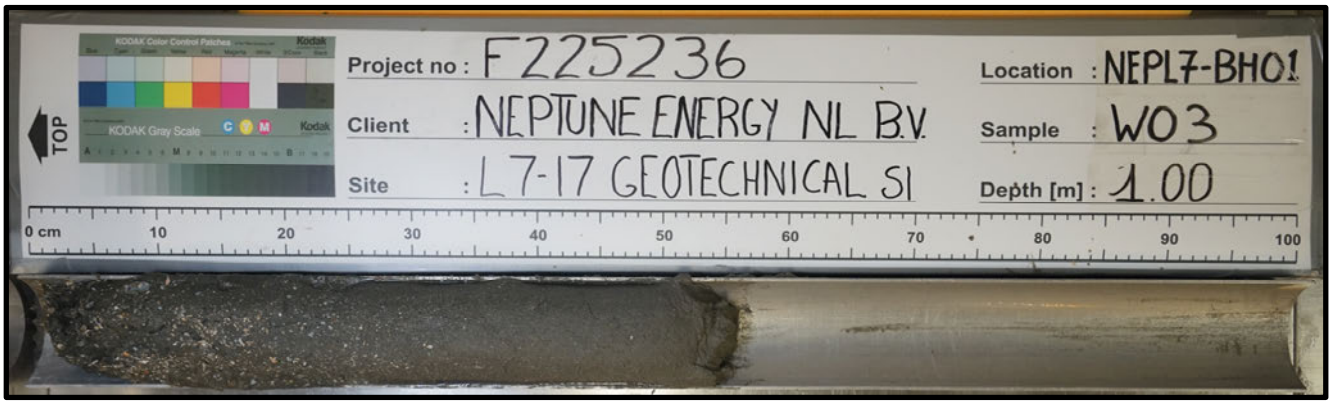
Location	:	NEPL7-BH01
Sample	:	W02
Depth [m BSF]	:	0.50
Note(s)	:	Intact



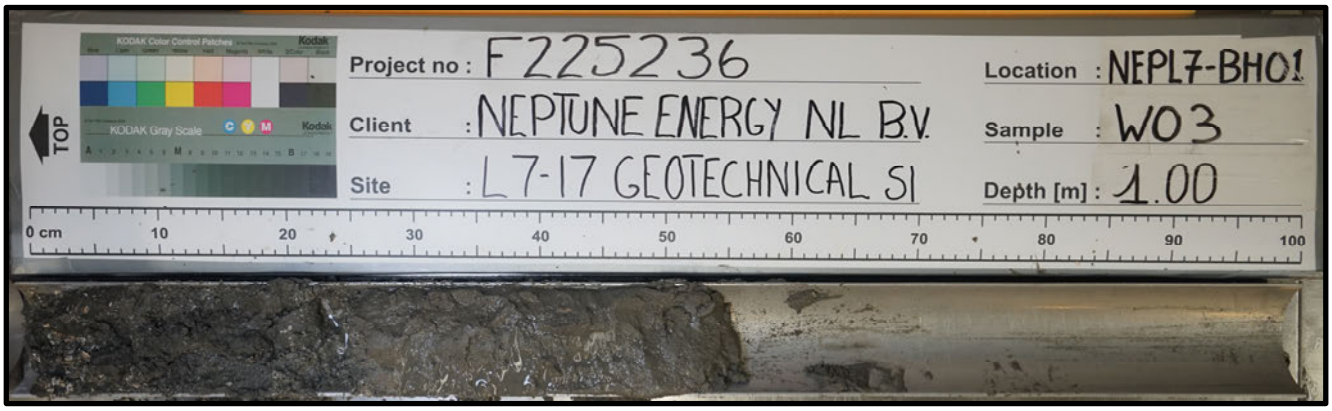
Location	:	NEPL7-BH01
Sample	:	W02
Depth [m BSF]	:	0.50
Note(s)	:	Split

SAMPLE PHOTOGRAPHS

Release Date: 6-3-2023
Drawn with: Photo_Plate_Maker v 6.0
Photo Folder: I Project Workfolders\Neptune L7\Photos\Proc
Presented Plate: Downhole_Portrait
Processed by: DBS
Date: 6-3-2023



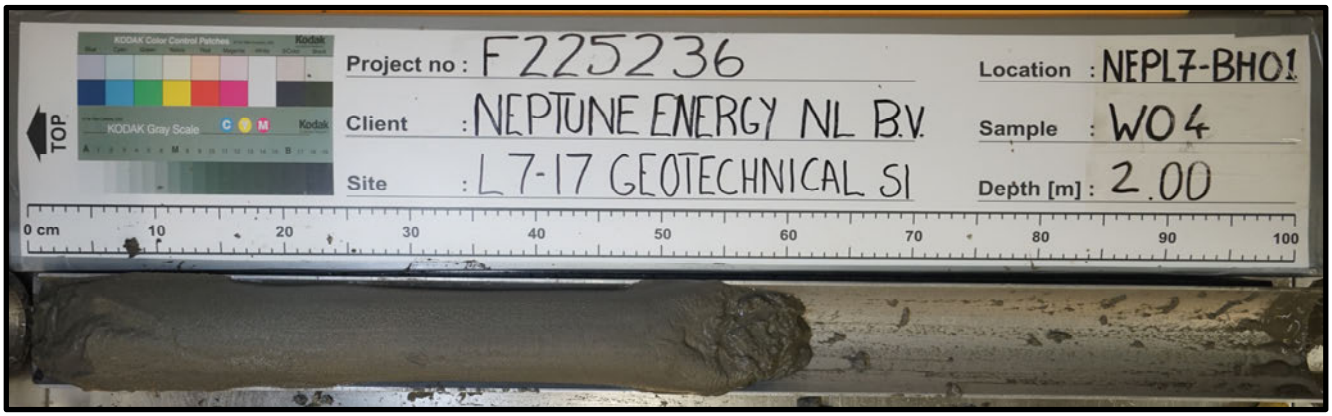
Location	:	NEPL7-BH01
Sample	:	W03
Depth [m BSF]	:	1.00
Note(s)	:	Intact



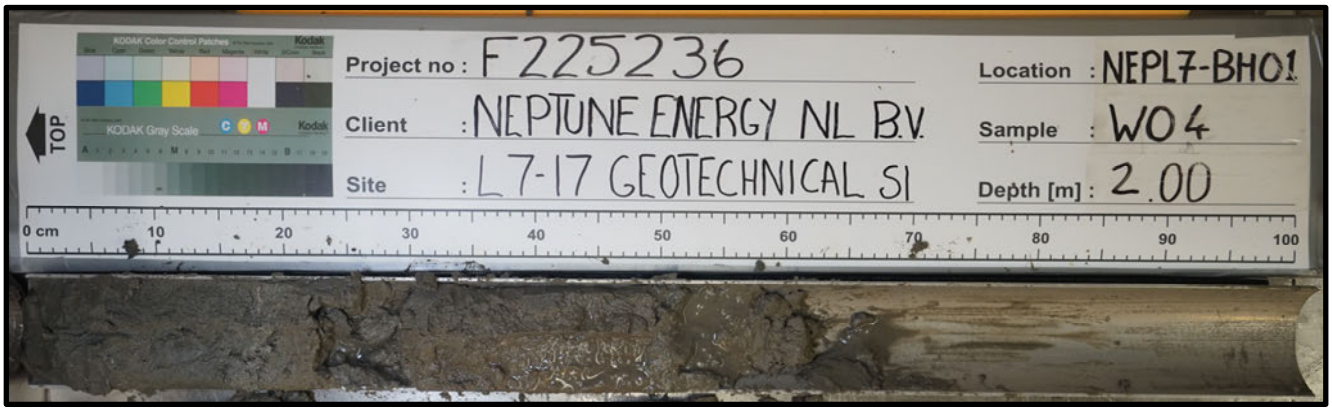
Location	:	NEPL7-BH01
Sample	:	W03
Depth [m BSF]	:	1.00
Note(s)	:	Split

SAMPLE PHOTOGRAPHS

Release Date: 6-3-2023
Drawn with: Photo_Plate_Maker v 6.0
Photo Folder: I Project Workfolders\Neptune L7\Photos\Proc
Presented Plate: Downhole_Portrait
Processed by: DBS
Date: 6-3-2023



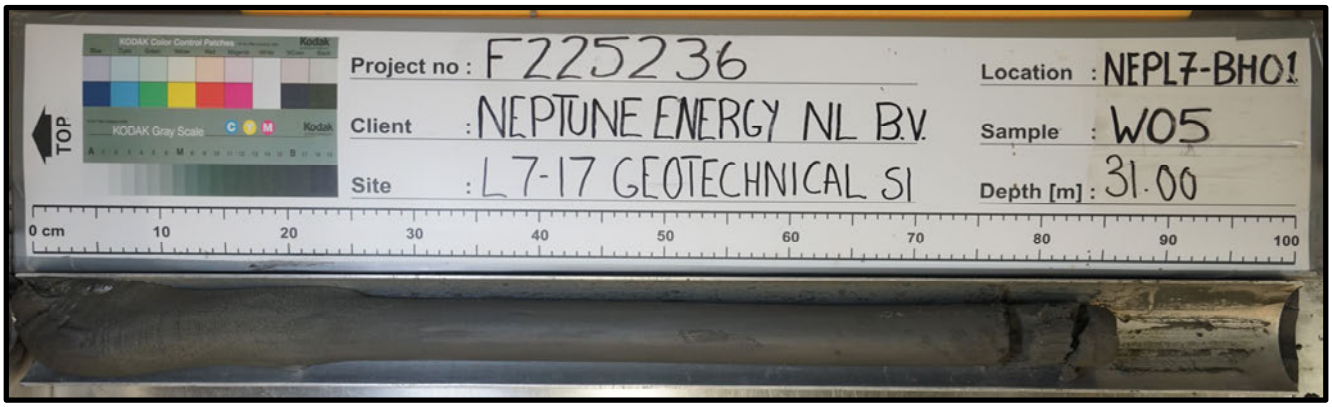
Location	:	NEPL7-BH01
Sample	:	W04
Depth [m BSF]	:	2.00
Note(s)	:	Intact



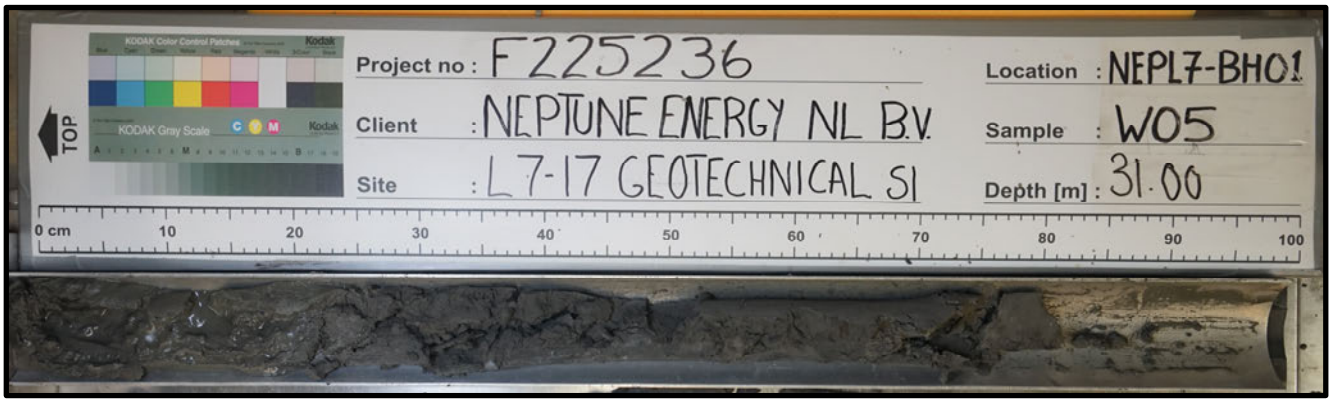
Location	:	NEPL7-BH01
Sample	:	W04
Depth [m BSF]	:	2.00
Note(s)	:	Split

SAMPLE PHOTOGRAPHS

Release Date: 6-3-2023
Drawn with: Photo_Plate_Maker v 6.0
Photo Folder: I Project Workfolders\Neptune L7\Photos\Proc
Presented Plate: Downhole_Portrait
Processed by: DBS
Date: 6-3-2023



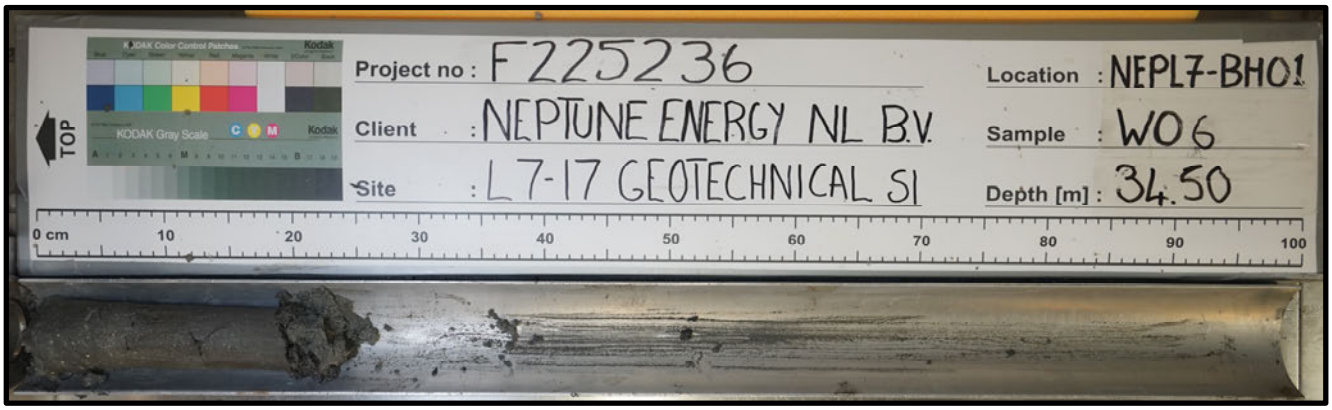
Location	:	NEPL7-BH01
Sample	:	W05
Depth [m BSF]	:	31.00
Note(s)	:	Intact



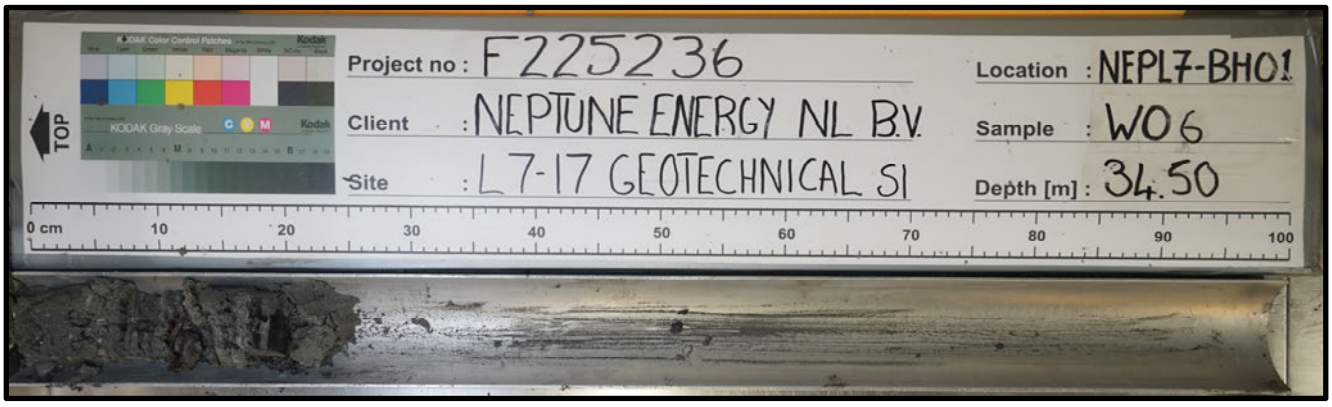
Location	:	NEPL7-BH01
Sample	:	W05
Depth [m BSF]	:	31.00
Note(s)	:	Split

SAMPLE PHOTOGRAPHS

Release Date: 6-3-2023
Drawn with: Photo_Plate_Maker v 6.0
Photo Folder: I Project Workfolders\Neptune L7\Photos\Proc
Presented Plate: Downhole_Portrait
Processed by: DBS
Date: 6-3-2023

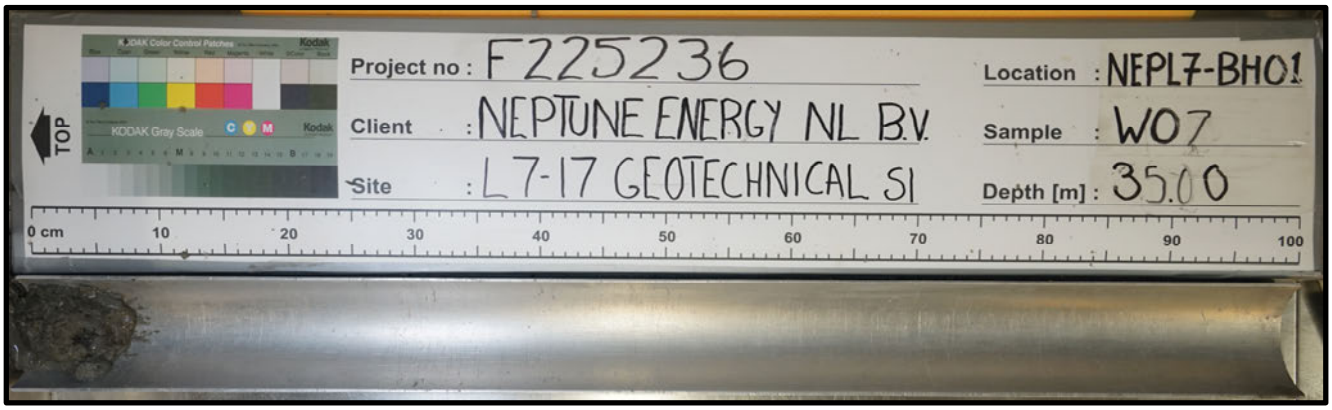


Location	:	NEPL7-BH01
Sample	:	W06
Depth [m BSF]	:	34.50
Note(s)	:	Intact

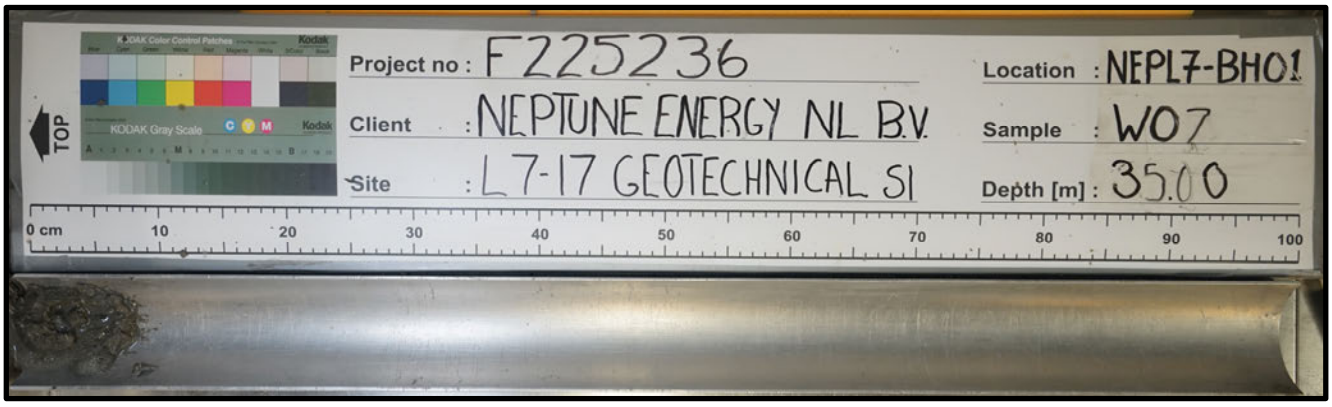


Location	:	NEPL7-BH01
Sample	:	W06
Depth [m BSF]	:	34.50
Note(s)	:	Split

SAMPLE PHOTOGRAPHS



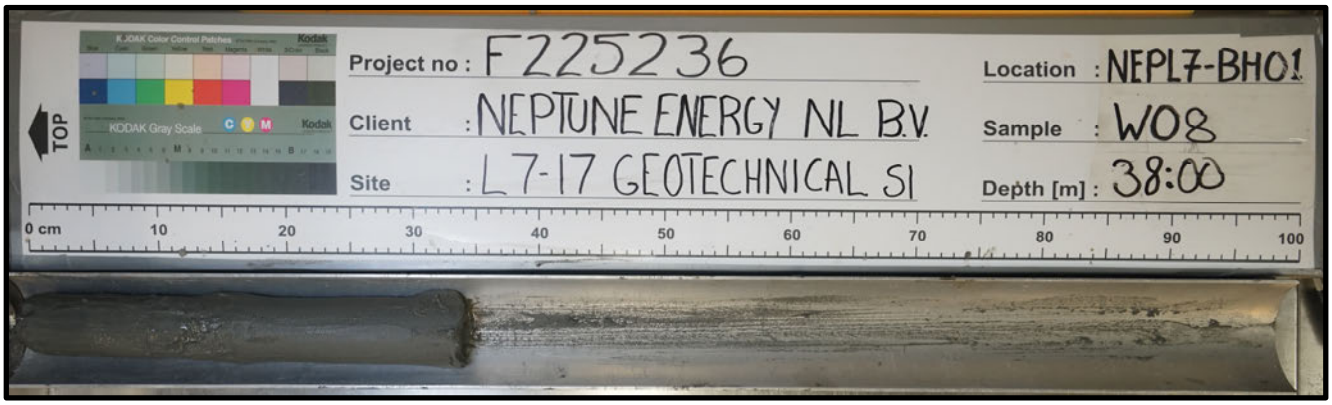
Location	:	NEPL7-BH01
Sample	:	W07
Depth [m BSF]	:	35.00
Note(s)	:	Intact



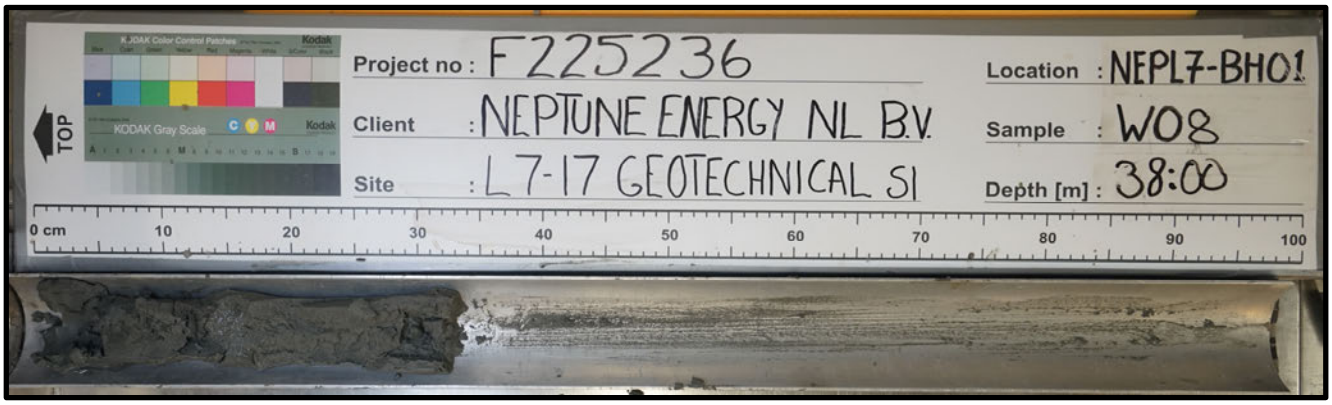
Location	:	NEPL7-BH01
Sample	:	W07
Depth [m BSF]	:	35.00
Note(s)	:	Split

SAMPLE PHOTOGRAPHS

Release Date: 6-3-2023
Drawn with: Photo_Plate_Maker v 6.0
Photo Folder: I Project Workfolders\Neptune L7\Photos\Proc
Presented Plate: Downhole_Portrait
Processed by: DBS
Date: 6-3-2023



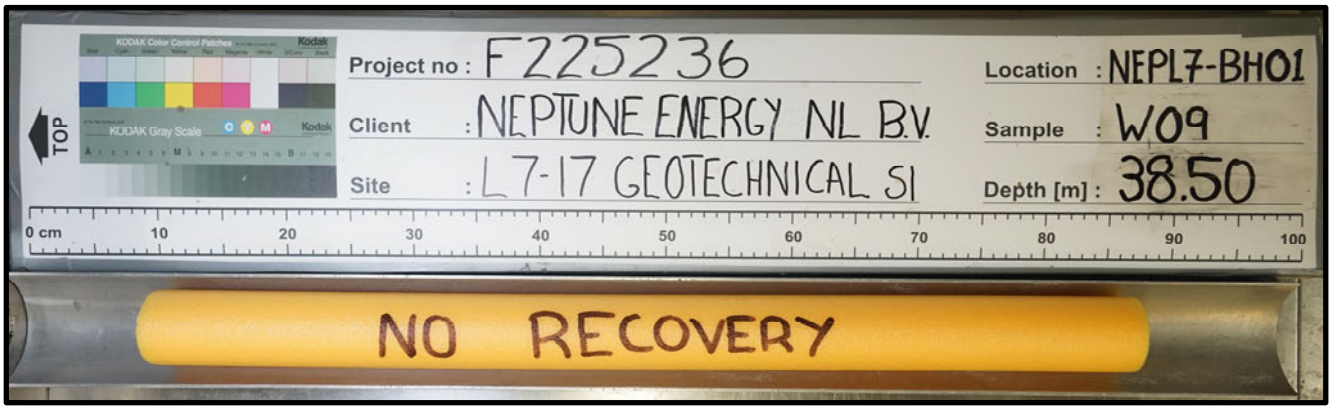
Location	:	NEPL7-BH01
Sample	:	W08
Depth [m BSF]	:	38.00
Note(s)	:	Intact



Location	:	NEPL7-BH01
Sample	:	W08
Depth [m BSF]	:	38.00
Note(s)	:	Split

SAMPLE PHOTOGRAPHS

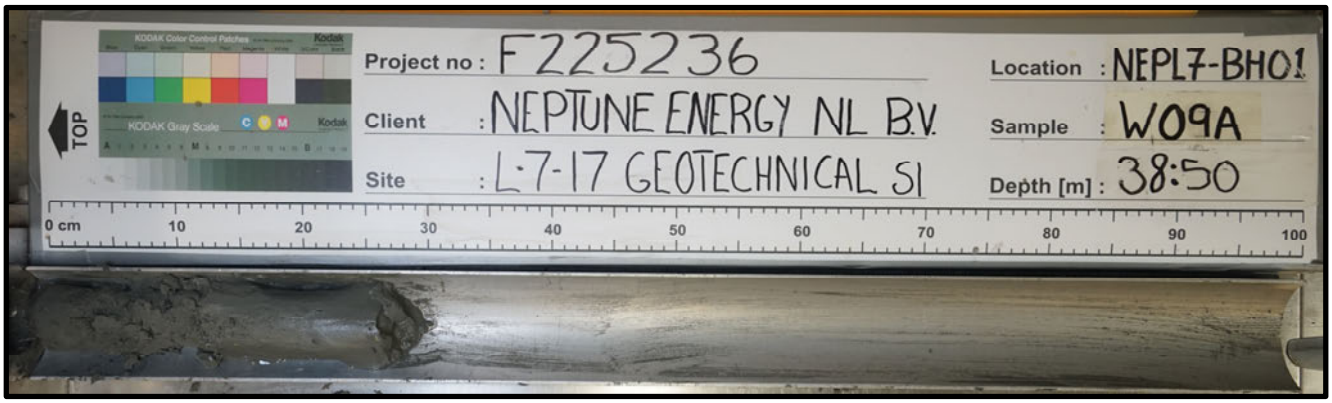
Release Date: 6-3-2023
Drawn with: Photo_Plate_Maker v 6.0
Photo Folder: I Project Workfolders\Neptune L7\Photos\Proc
Presented Plate: Downhole_Portrait
Processed by: DBS
Date: 6-3-2023



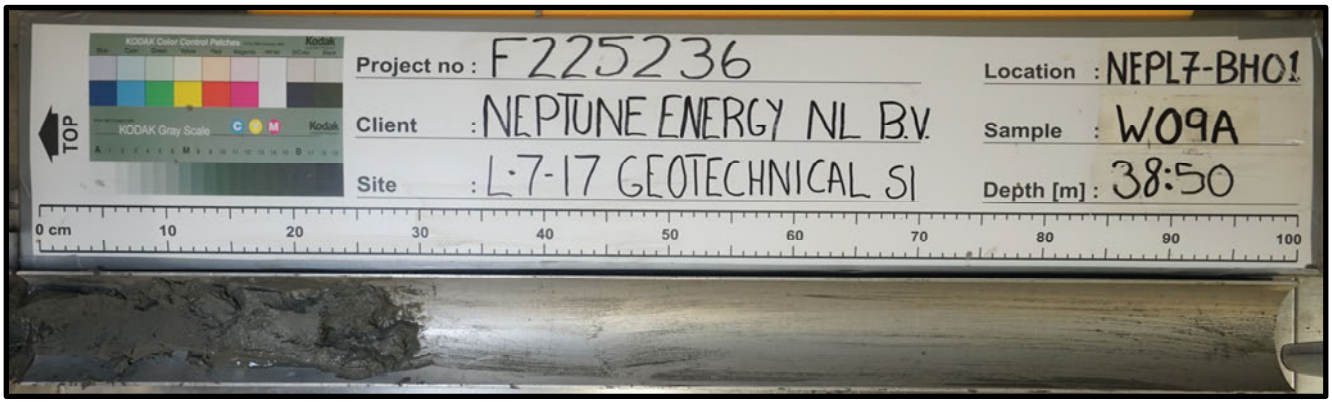
Location	:	NEPL7-BH01
Sample	:	W09
Depth [m BSF]	:	38.50
Note(s)	:	No recovery

Release Date: 6-3-2023
Drawn with: Photo_Plate_Maker v 6.0
Photo Folder: I Project Workfolders\Neptune L7\Photos\Proc
Presented Plate: Downhole_Portrait
Processed by: DBS
Date: 6-3-2023

SAMPLE PHOTOGRAPHS



Location	:	NEPL7-BH01
Sample	:	W09A
Depth [m BSF]	:	38.50
Note(s)	:	Intact

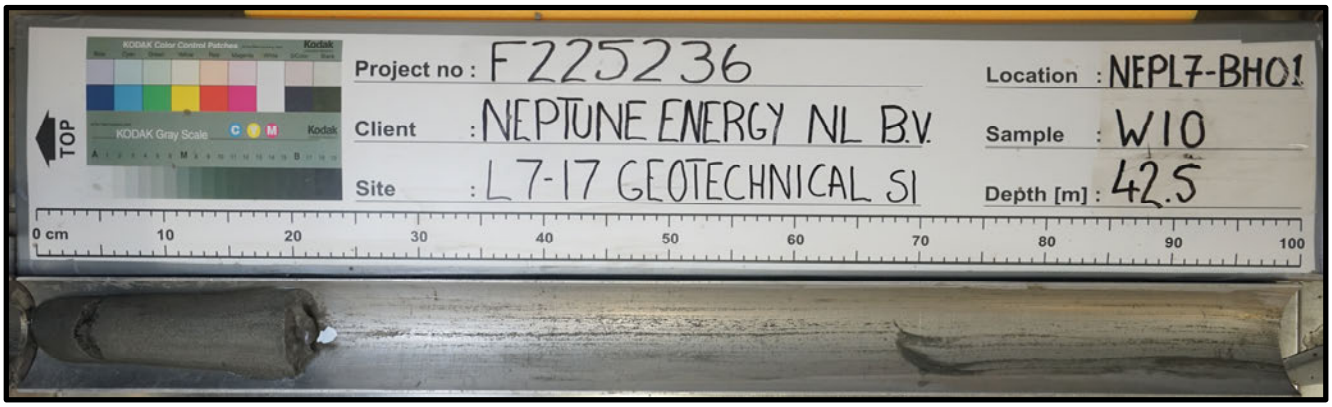


Location	:	NEPL7-BH01
Sample	:	W09A
Depth [m BSF]	:	38.50
Note(s)	:	Split

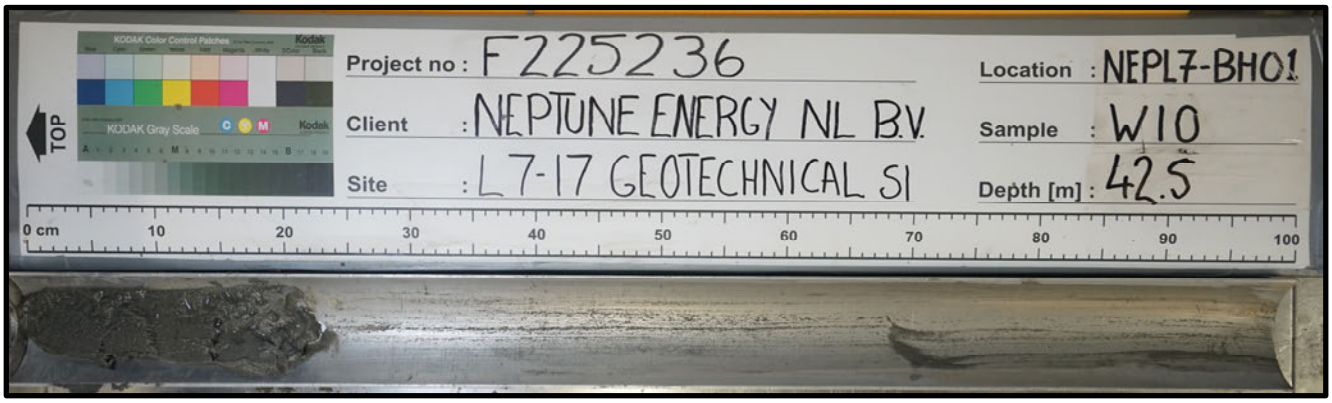
SAMPLE PHOTOGRAPHS

Release Date: 6-3-2023
Drawn with: Photo_Plate_Maker v 6.0
Photo Folder: I Project Workfolders\Neptune L7\Photos\Proc
Presented Plate: Downhole_Portrait
Processed by: DBS
Date: 6-3-2023

Release Date: 6-3-2023
Drawn with: Photo_Plate_Maker v 6.0
Photo Folder: I Project Workfolders\Neptune L7\Photos\Proc
Presented Plate: Downhole_Portrait
Processed by: DBS
Date: 6-3-2023

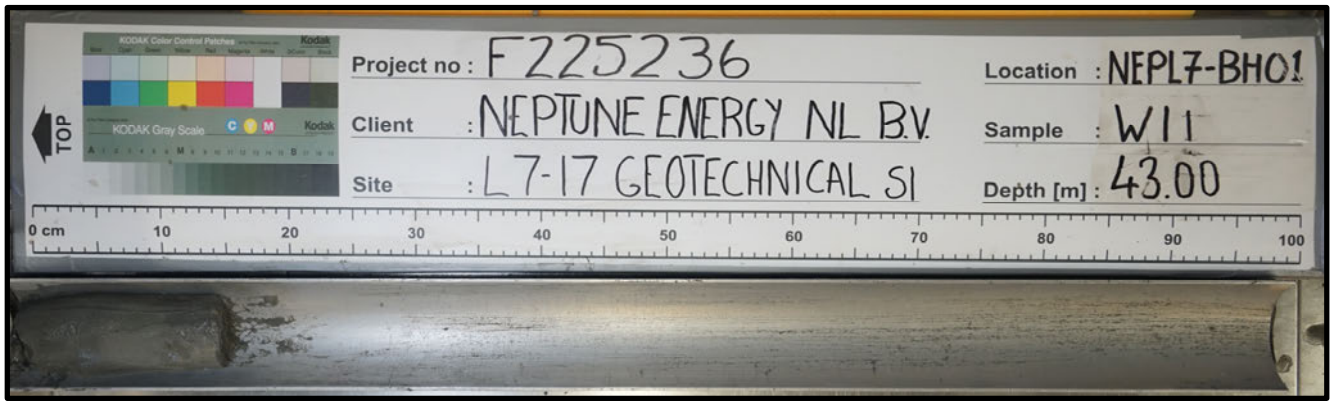


Location	:	NEPL7-BH01
Sample	:	W10
Depth [m BSF]	:	42.50
Note(s)	:	Intact

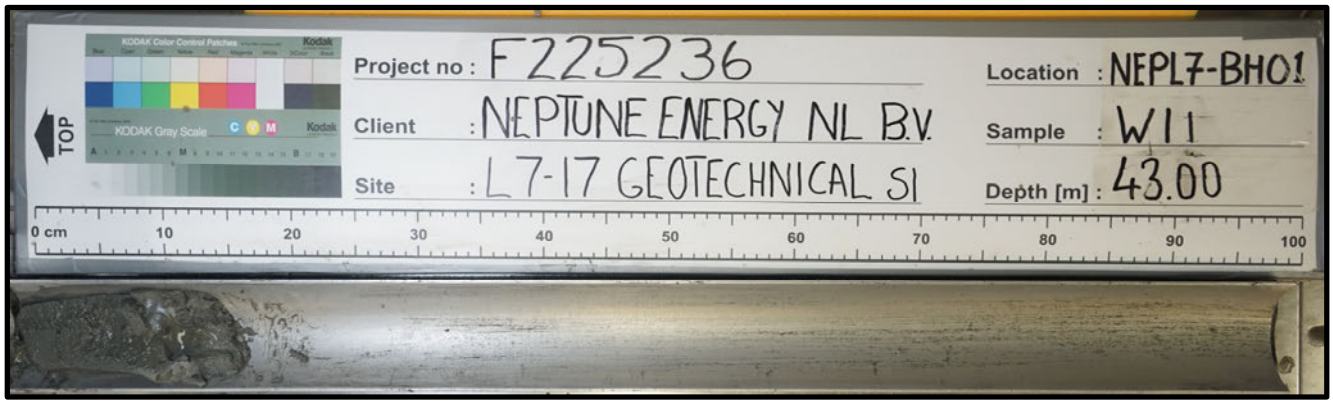


Location	:	NEPL7-BH01
Sample	:	W10
Depth [m BSF]	:	42.50
Note(s)	:	Split

SAMPLE PHOTOGRAPHS



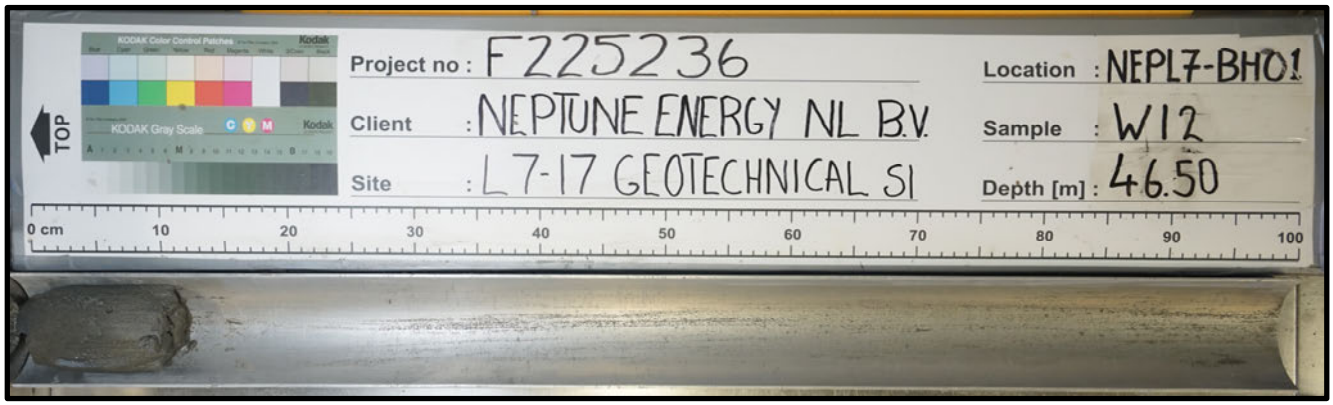
Location	:	NEPL7-BH01
Sample	:	W11
Depth [m BSF]	:	43.00
Note(s)	:	Intact



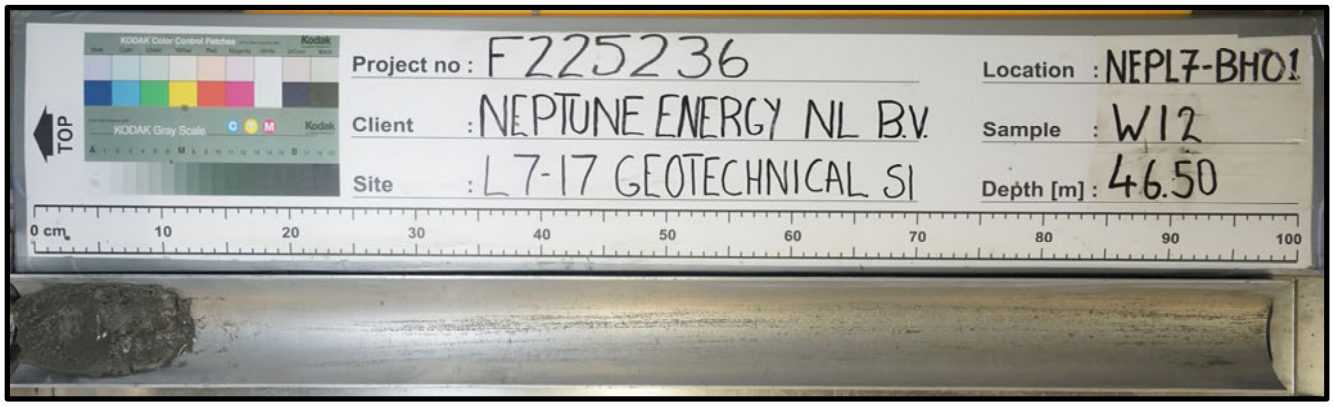
Location	:	NEPL7-BH01
Sample	:	W11
Depth [m BSF]	:	43.00
Note(s)	:	Split

SAMPLE PHOTOGRAPHS

Release Date: 6-3-2023
Drawn with: Photo_Plate_Maker v 6.0
Photo Folder: I Project Workfolders\Neptune L7\Photos\Proc
Presented Plate: Downhole_Portrait
Processed by: DBS
Date: 6-3-2023



Location	:	NEPL7-BH01
Sample	:	W12
Depth [m BSF]	:	46.50
Note(s)	:	Intact

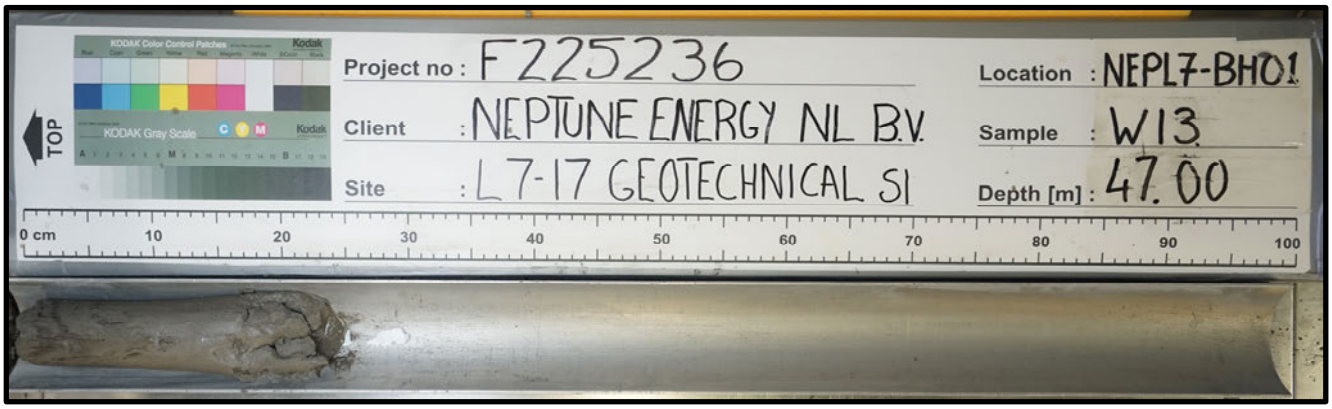


Location	:	NEPL7-BH01
Sample	:	W12
Depth [m BSF]	:	46.50
Note(s)	:	Split

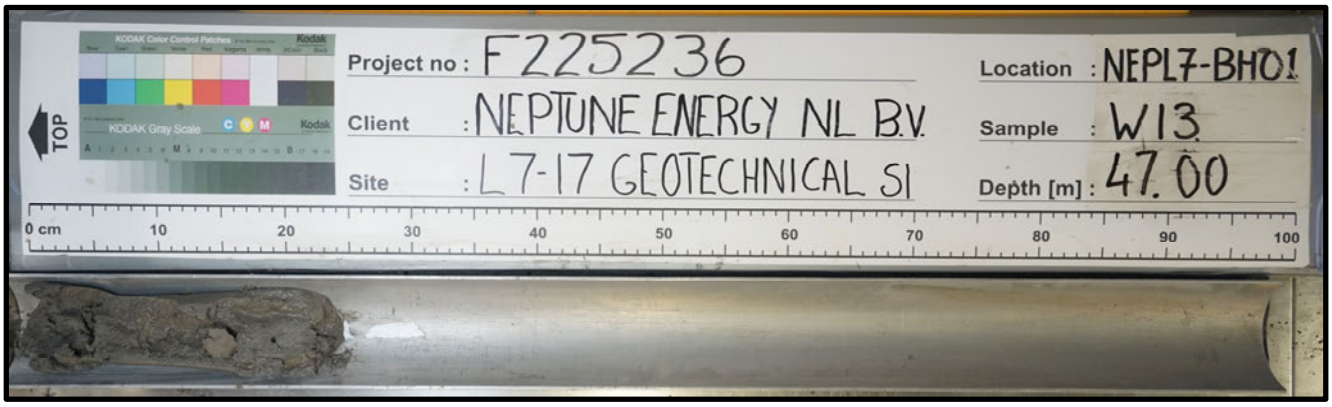
SAMPLE PHOTOGRAPHS

Release Date: 6-3-2023
Drawn with: Photo_Plate_Maker v 6.0
Photo Folder: I Project Workfolders\Neptune L7\Photos\Proc
Presented Plate: Downhole_Portrait
Processed by: DBS
Date: 6-3-2023

Release Date: 6-3-2023
Drawn with: Photo_Plate_Maker v 6.0
Photo Folder: I Project Workfolders\Neptune L7\Photos\Proc
Presented Plate: Downhole_Portrait
Processed by: DBS
Date: 6-3-2023

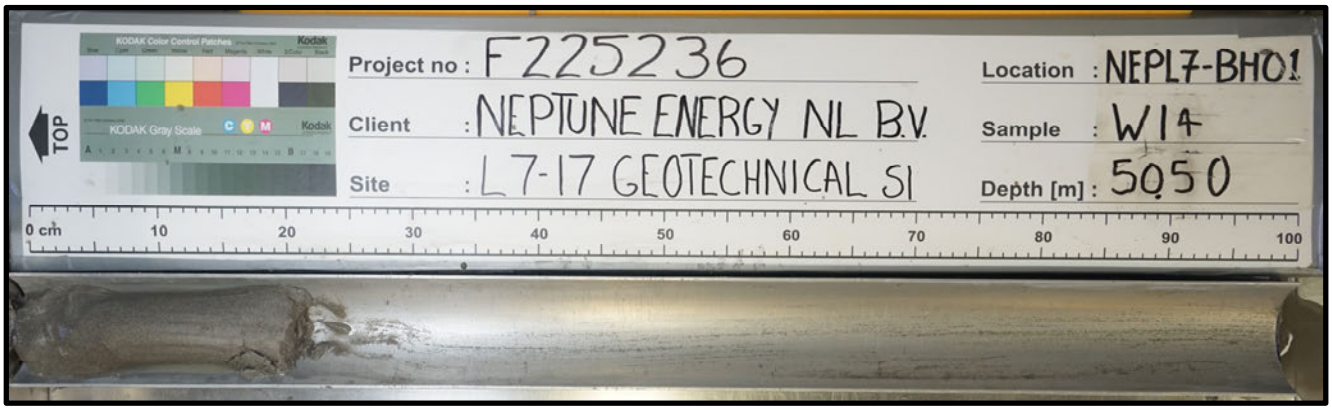


Location	:	NEPL7-BH01
Sample	:	W13
Depth [m BSF]	:	47.00
Note(s)	:	Intact

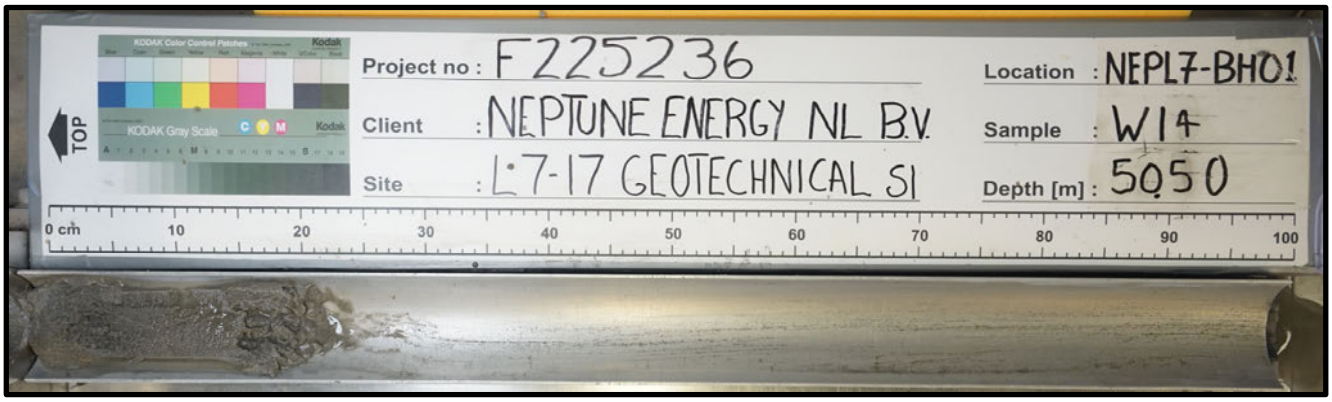


Location	:	NEPL7-BH01
Sample	:	W13
Depth [m BSF]	:	47.00
Note(s)	:	Split

SAMPLE PHOTOGRAPHS



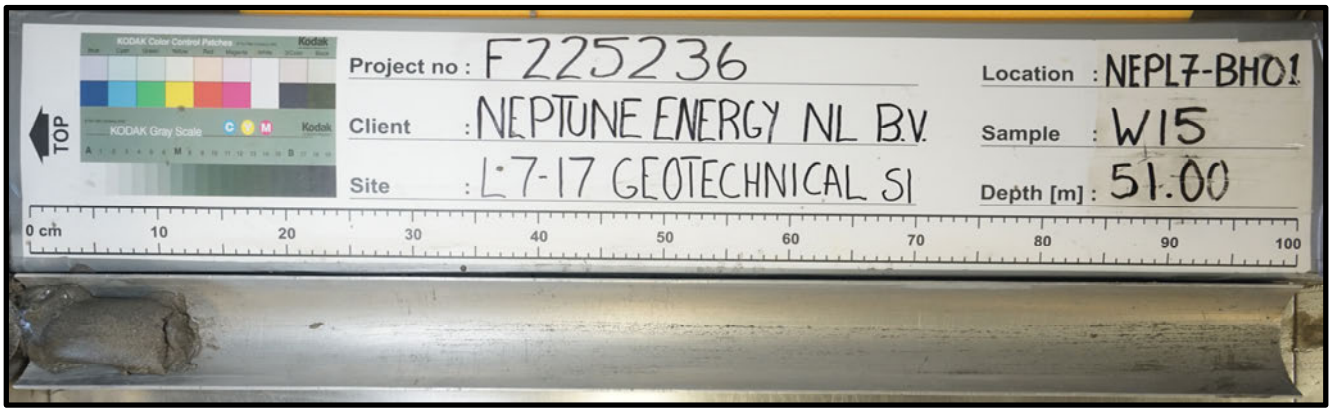
Location	:	NEPL7-BH01
Sample	:	W14
Depth [m BSF]	:	50.50
Note(s)	:	Intact



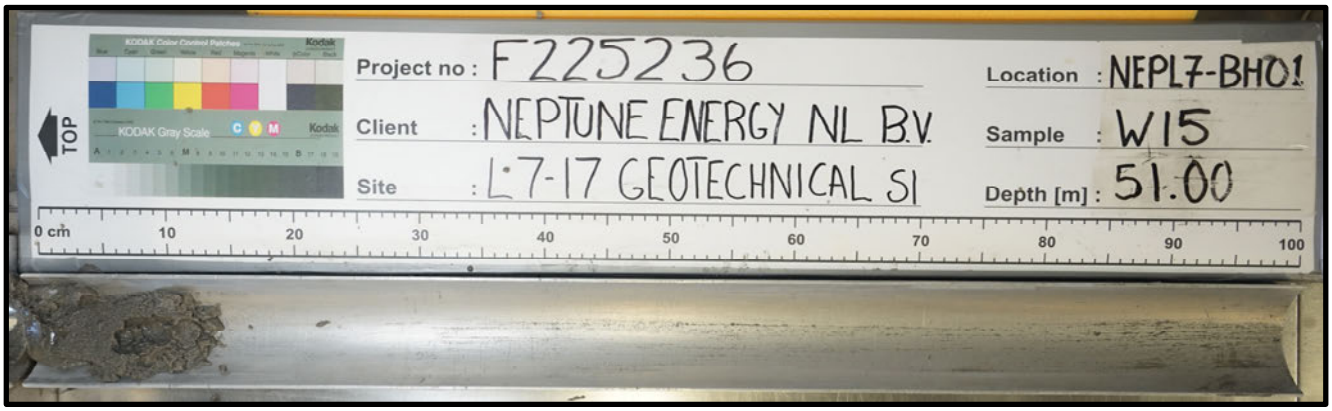
Location	:	NEPL7-BH01
Sample	:	W14
Depth [m BSF]	:	50.50
Note(s)	:	Split

SAMPLE PHOTOGRAPHS

Release Date: 6-3-2023
Drawn with: Photo_Plate_Maker v 6.0
Photo Folder: I Project Workfolders\Neptune L7\Photos\Proc
Presented Plate: Downhole_Portrait
Processed by: DBS
Date: 6-3-2023



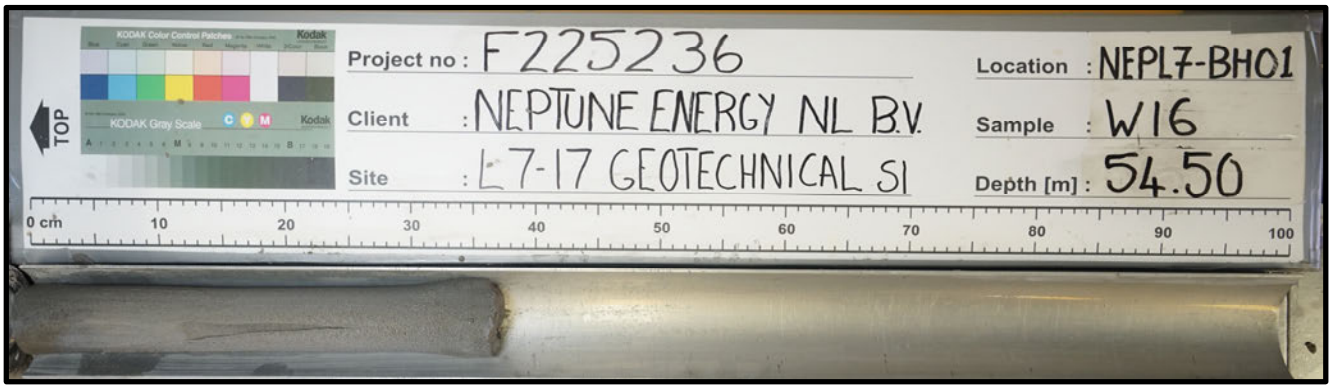
Location	:	NEPL7-BH01
Sample	:	W15
Depth [m BSF]	:	51.00
Note(s)	:	Intact



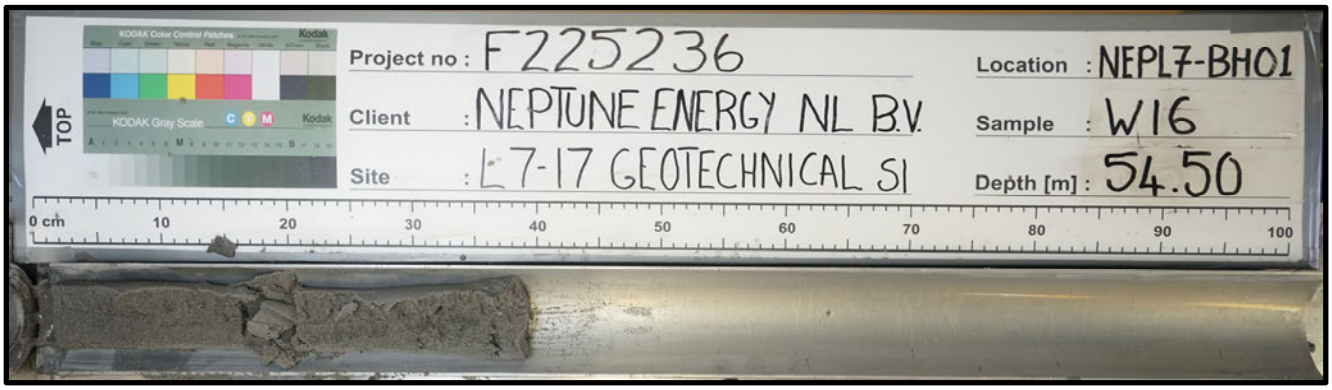
Location	:	NEPL7-BH01
Sample	:	W15
Depth [m BSF]	:	51.00
Note(s)	:	Split

SAMPLE PHOTOGRAPHS

Release Date: 6-3-2023
Drawn with: Photo_Plate_Maker v 6.0
Photo Folder: I Project Workfolders\Neptune L7\Photos\Proc
Presented Plate: Downhole_Portrait
Processed by: DBS
Date: 6-3-2023



Location	:	NEPL7-BH01
Sample	:	W16
Depth [m BSF]	:	54.50
Note(s)	:	Intact

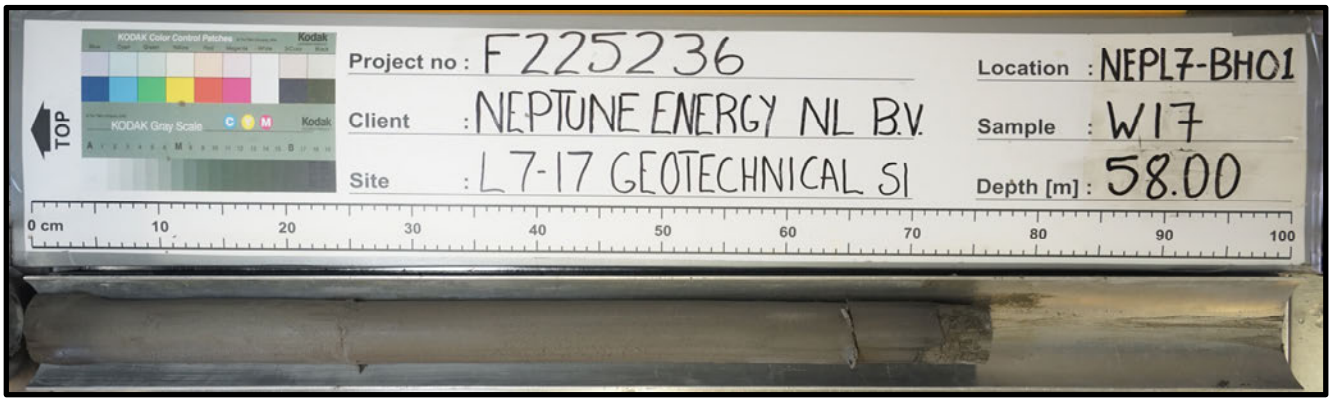


Location	:	NEPL7-BH01
Sample	:	W16
Depth [m BSF]	:	54.50
Note(s)	:	Split

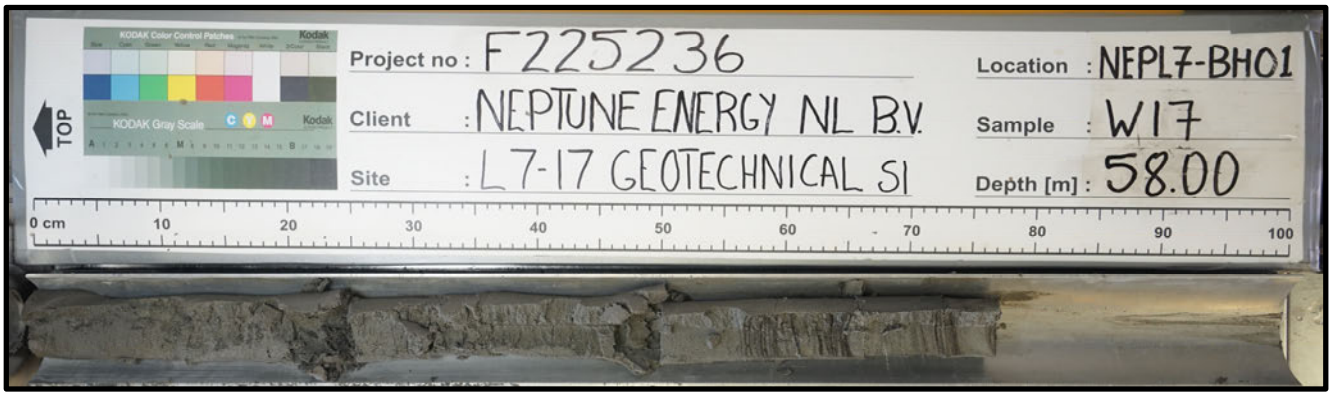
SAMPLE PHOTOGRAPHS

Release Date: 6-3-2023
Drawn with: Photo_Plate_Maker v 6.0
Photo Folder: I Project Workfolders\Neptune L7\Photos\Proc
Presented Plate: Downhole_Portrait
Processed by: DBS
Date: 6-3-2023

Release Date: 6-3-2023
Drawn with: Photo_Plate_Maker v 6.0
Photo Folder: I Project Workfolders\Neptune L7\Photos\Proc
Presented Plate: Downhole_Portrait
Processed by: DBS
Date: 6-3-2023



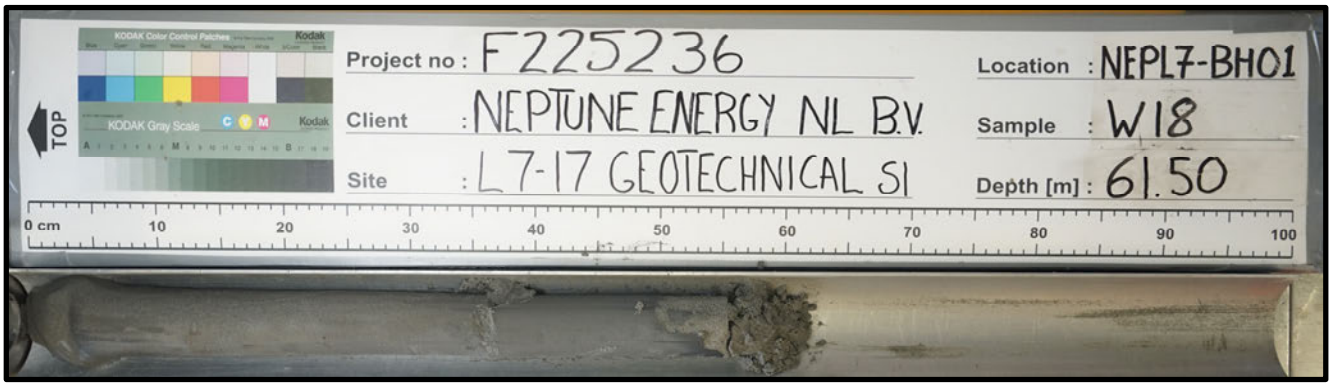
Location	:	NEPL7-BH01
Sample	:	W17
Depth [m BSF]	:	58.00
Note(s)	:	Intact



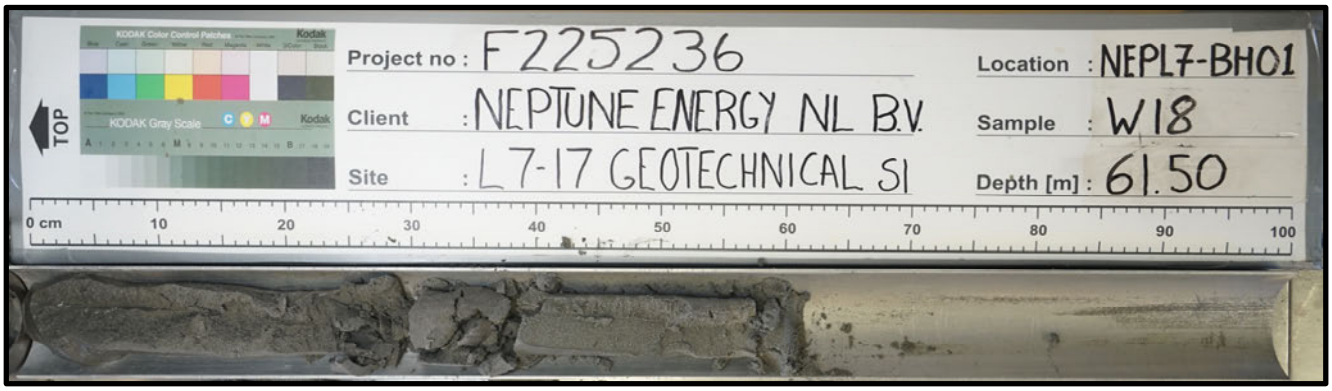
Location	:	NEPL7-BH01
Sample	:	W17
Depth [m BSF]	:	58.00
Note(s)	:	Split

SAMPLE PHOTOGRAPHS

Release Date: 6-3-2023
Drawn with: Photo_Plate_Maker v 6.0
Photo Folder: I Project Workfolders\Neptune L7\Photos\Proc
Presented Plate: Downhole_Portrait
Processed by: DBS
Date: 6-3-2023



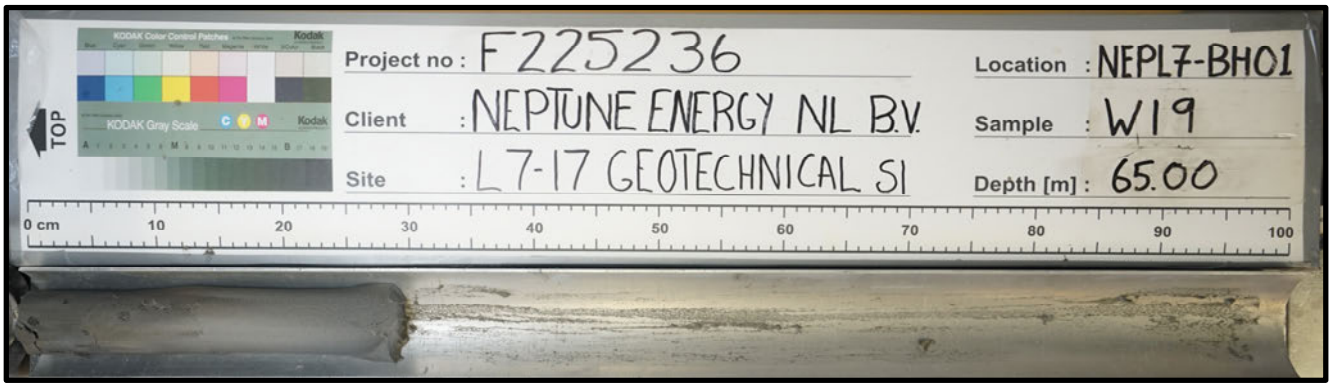
Location	:	NEPL7-BH01
Sample	:	W18
Depth [m BSF]	:	61.50
Note(s)	:	Intact



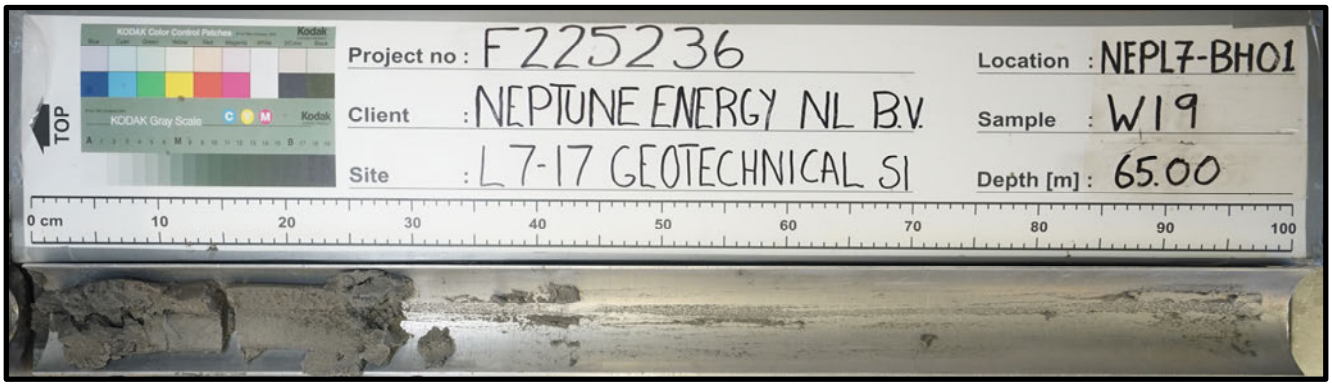
Location	:	NEPL7-BH01
Sample	:	W18
Depth [m BSF]	:	61.50
Note(s)	:	Split

SAMPLE PHOTOGRAPHS

Release Date: 6-3-2023
Drawn with: Photo_Plate_Maker v 6.0
Photo Folder: I Project Workfolders\Neptune L7\Photos\Proc
Presented Plate: Downhole_Portrait
Processed by: DBS
Date: 6-3-2023



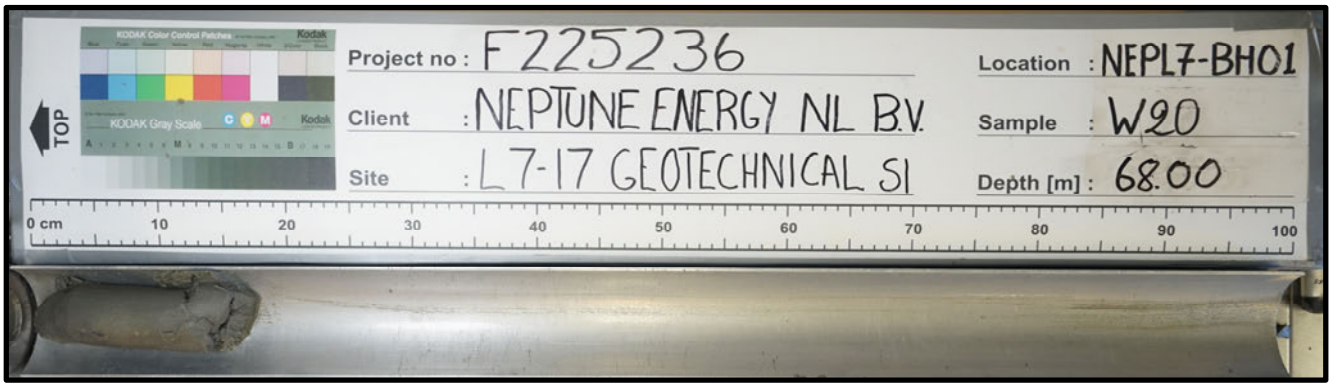
Location	:	NEPL7-BH01
Sample	:	W19
Depth [m BSF]	:	65.00
Note(s)	:	Intact



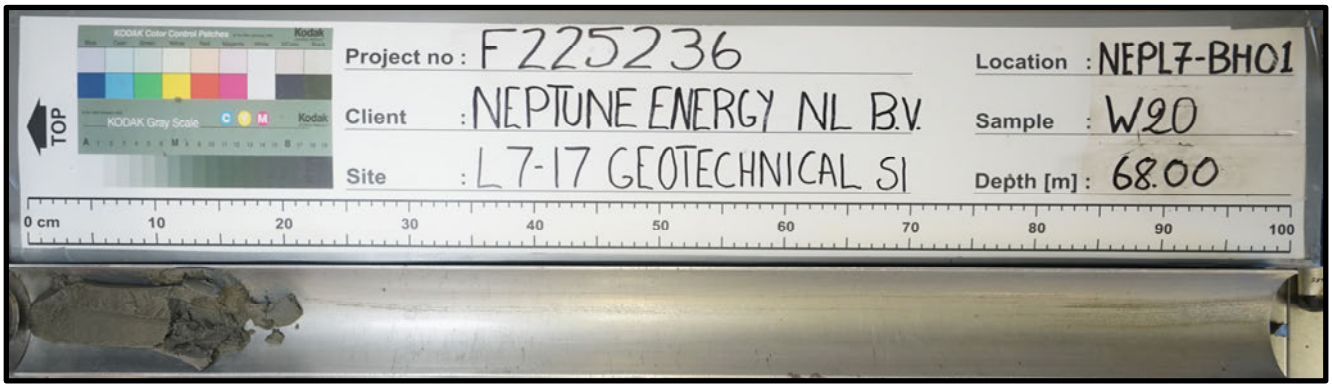
Location	:	NEPL7-BH01
Sample	:	W19
Depth [m BSF]	:	65.00
Note(s)	:	Split

SAMPLE PHOTOGRAPHS

Release Date: 6-3-2023
Drawn with: Photo_Plate_Maker v 6.0
Photo Folder: I Project Workfolders\Neptune L7\Photos\Proc
Presented Plate: Downhole_Portrait
Processed by: DBS
Date: 6-3-2023



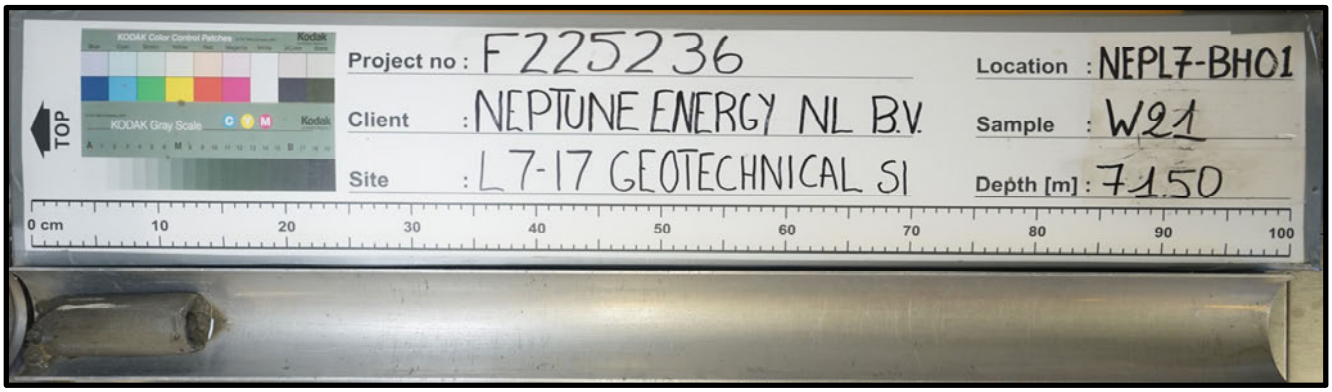
Location	:	NEPL7-BH01
Sample	:	W20
Depth [m BSF]	:	68.00
Note(s)	:	Intact



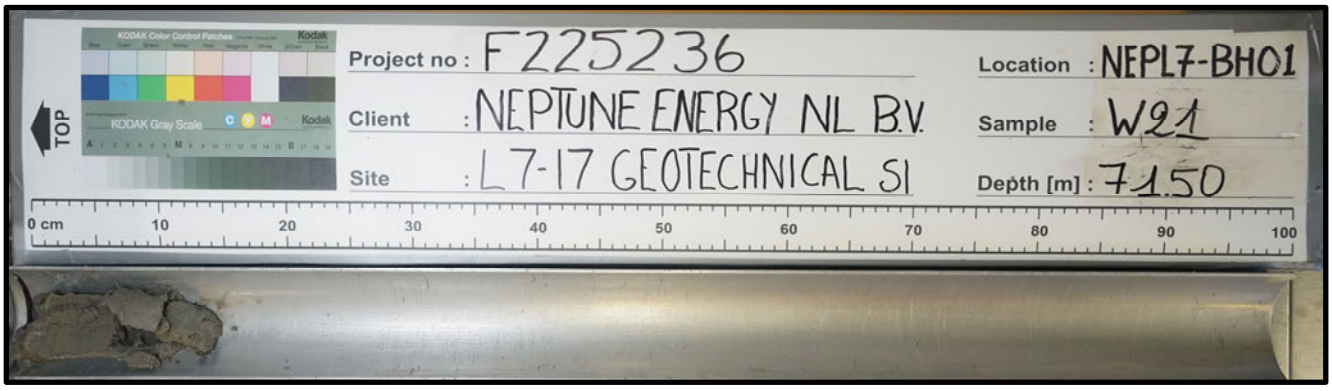
Location	:	NEPL7-BH01
Sample	:	W20
Depth [m BSF]	:	68.00
Note(s)	:	Split

SAMPLE PHOTOGRAPHS

Release Date: 6-3-2023
Drawn with: Photo_Plate_Maker v 6.0
Photo Folder: I Project Workfolders\Neptune L7\Photos\Proc
Presented Plate: Downhole_Portrait
Processed by: DBS
Date: 6-3-2023



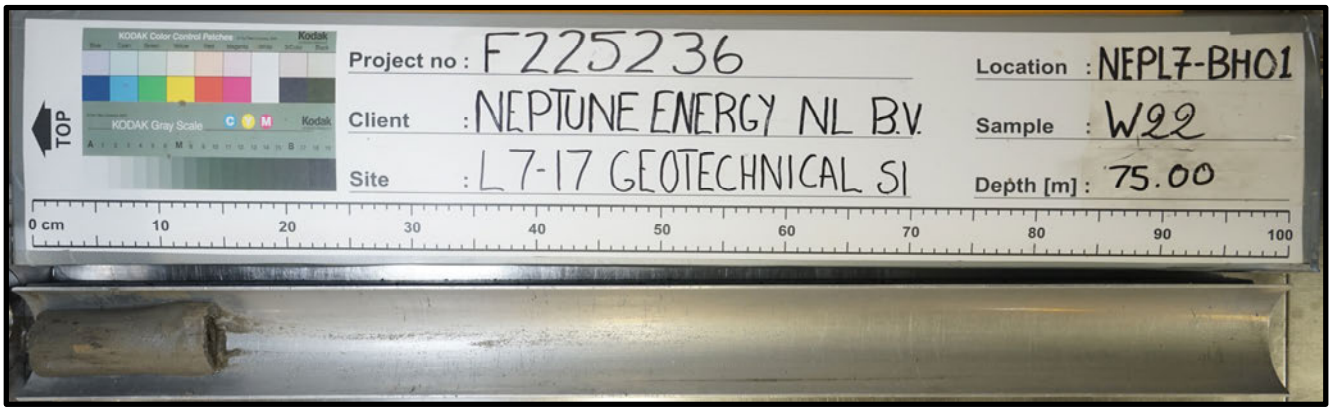
Location	:	NEPL7-BH01
Sample	:	W21
Depth [m BSF]	:	71.50
Note(s)	:	Intact



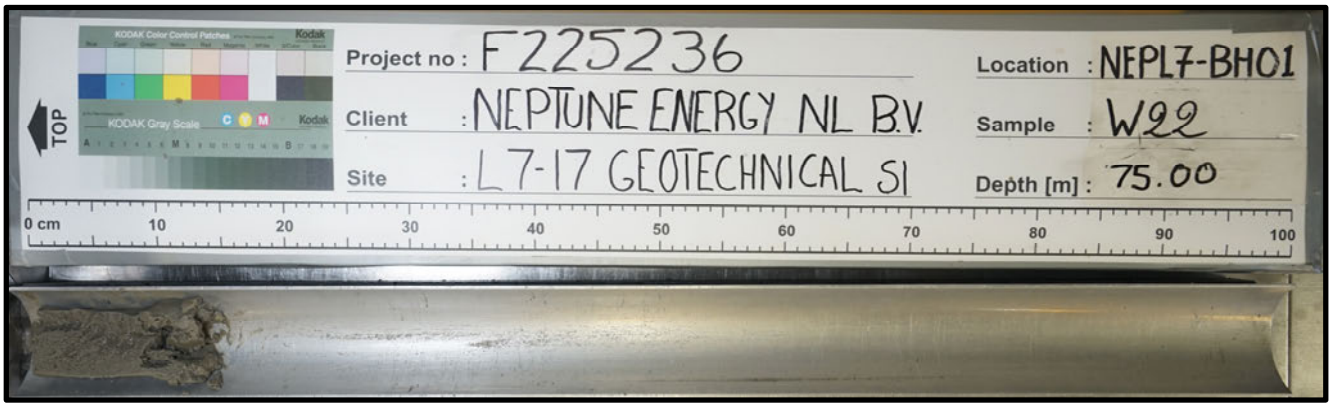
Location	:	NEPL7-BH01
Sample	:	W21
Depth [m BSF]	:	71.50
Note(s)	:	Split

SAMPLE PHOTOGRAPHS

Release Date: 6-3-2023
Drawn with: Photo_Plate_Maker v 6.0
Photo Folder: I Project Workfolders\Neptune L7\Photos\Proc
Presented Plate: Downhole_Portrait
Processed by: DBS
Date: 6-3-2023

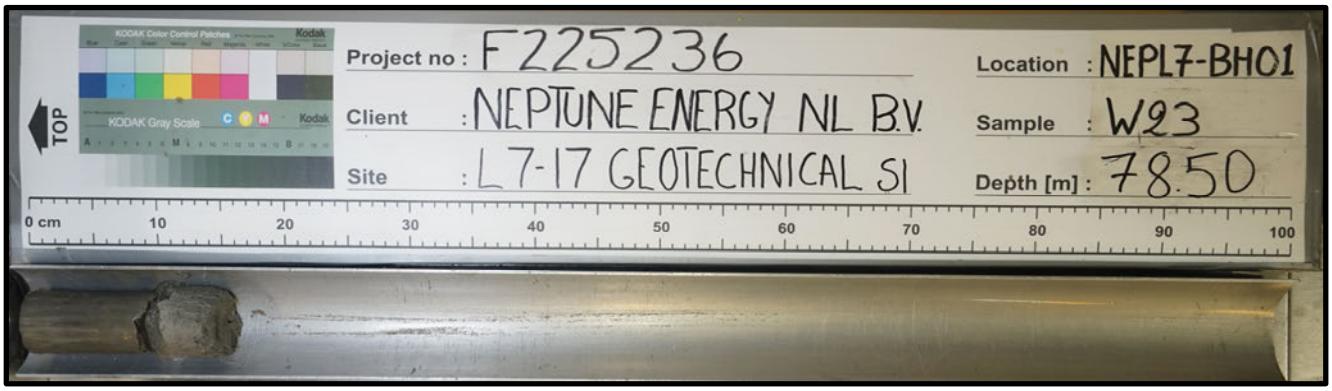


Location	:	NEPL7-BH01
Sample	:	W22
Depth [m BSF]	:	75.00
Note(s)	:	Intact



Location	:	NEPL7-BH01
Sample	:	W22
Depth [m BSF]	:	75.00
Note(s)	:	Split

SAMPLE PHOTOGRAPHS



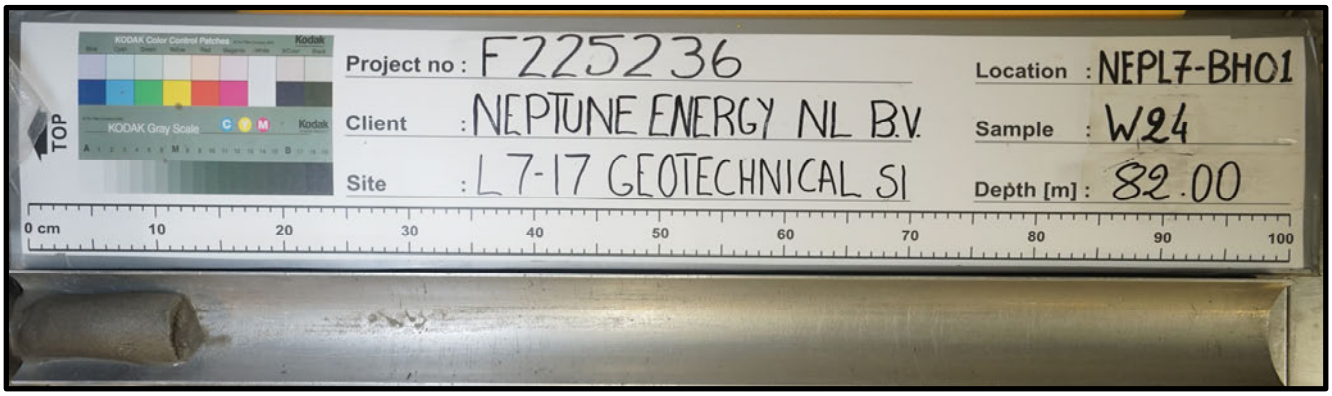
Location	:	NEPL7-BH01
Sample	:	W23
Depth [m BSF]	:	78.50
Note(s)	:	Intact

No photograph available

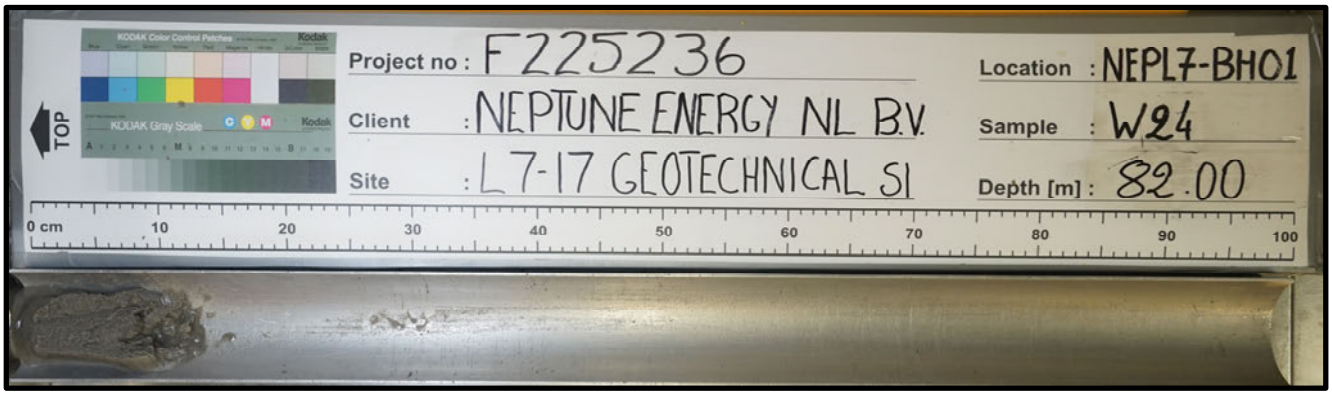
Location	:	NEPL7-BH01
Sample	:	W23
Depth [m BSF]	:	78.50

SAMPLE PHOTOGRAPHS

Release Date: 6-3-2023
Drawn with: Photo_Plate_Maker v 6.0
Photo Folder: I Project Workfolders\Neptune L7\Photos\Proc
Presented Plate: Downhole_Portrait
Processed by: DBS
Date: 6-3-2023



Location	:	NEPL7-BH01
Sample	:	W24
Depth [m BSF]	:	82.00
Note(s)	:	Intact

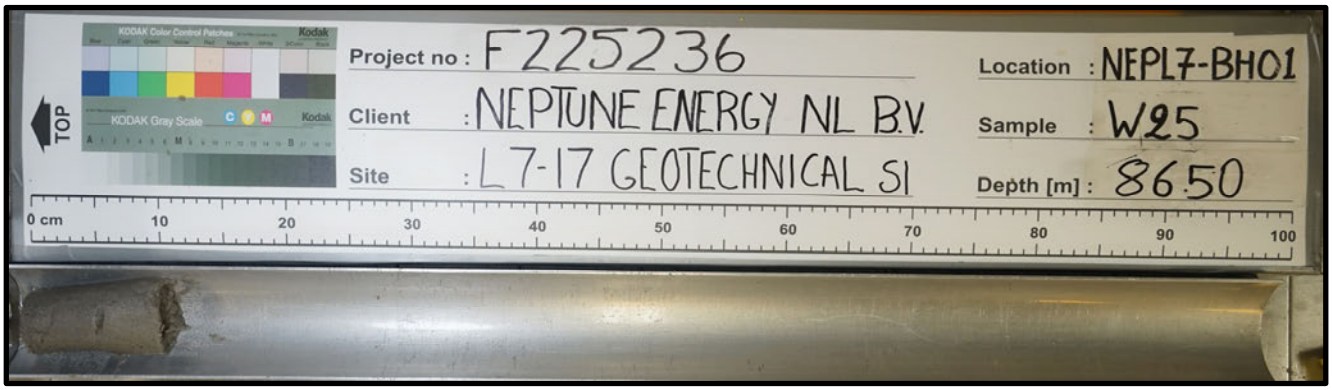


Location	:	NEPL7-BH01
Sample	:	W24
Depth [m BSF]	:	82.00
Note(s)	:	Split

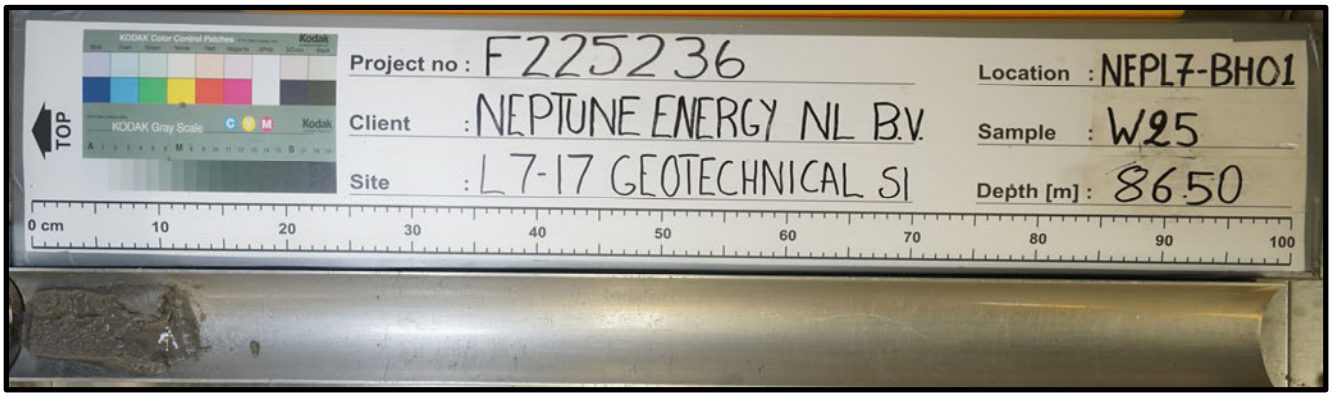
SAMPLE PHOTOGRAPHS

Release Date: 6-3-2023
Drawn with: Photo_Plate_Maker v 6.0
Photo Folder: I Project Workfolders\Neptune L7\Photos\Proc
Presented Plate: Downhole_Portrait
Processed by: DBS
Date: 6-3-2023

Release Date: 6-3-2023
Drawn with: Photo_Plate_Maker v 6.0
Photo Folder: I Project Workfolders\Neptune L7\Photos\Proc
Presented Plate: Downhole_Portrait
Processed by: DBS
Date: 6-3-2023

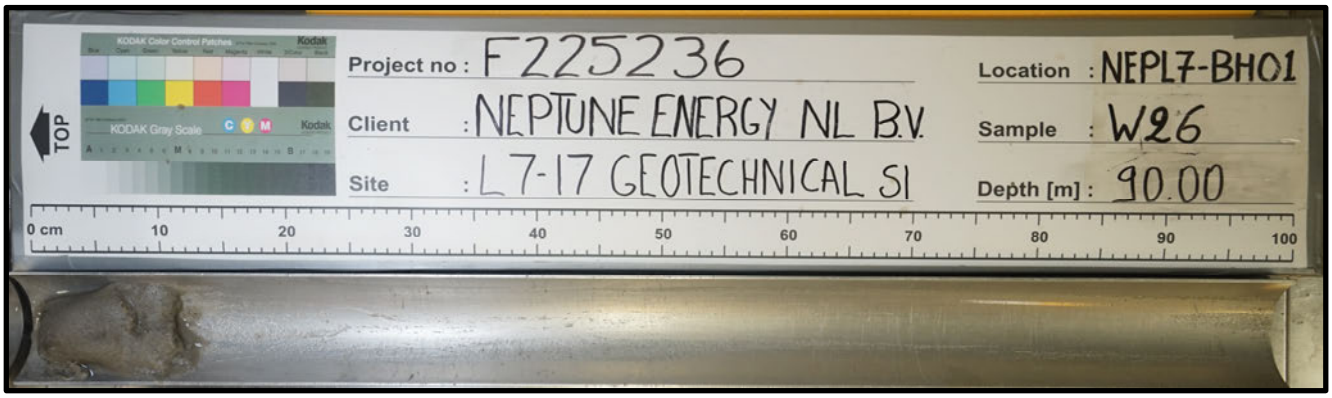


Location	:	NEPL7-BH01
Sample	:	W25
Depth [m BSF]	:	86.50
Note(s)	:	Intact

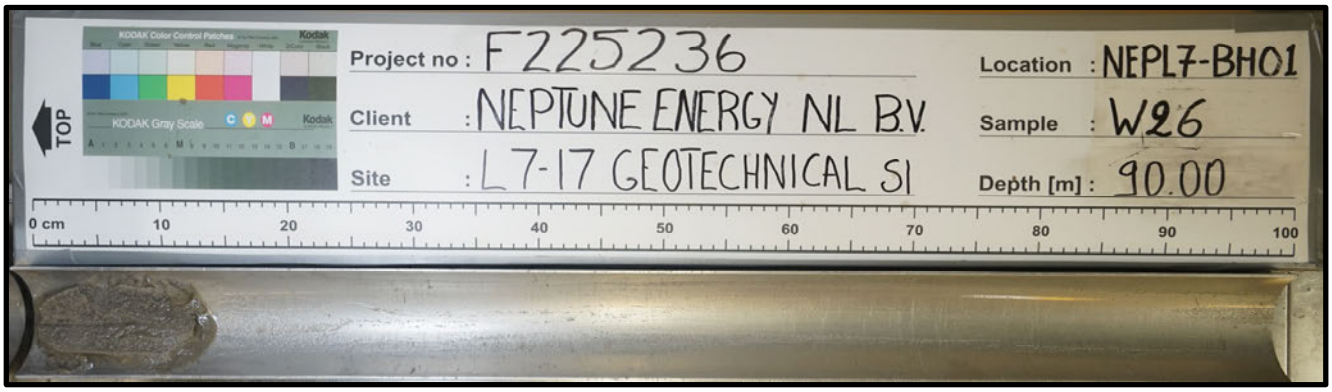


Location	:	NEPL7-BH01
Sample	:	W25
Depth [m BSF]	:	86.50
Note(s)	:	Split

SAMPLE PHOTOGRAPHS



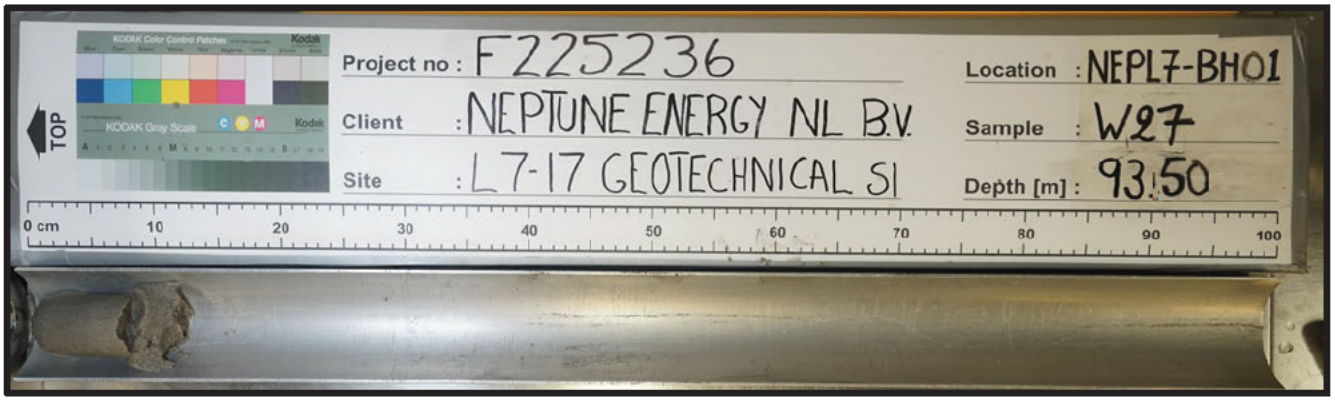
Location	:	NEPL7-BH01
Sample	:	W26
Depth [m BSF]	:	90.00
Note(s)	:	Intact



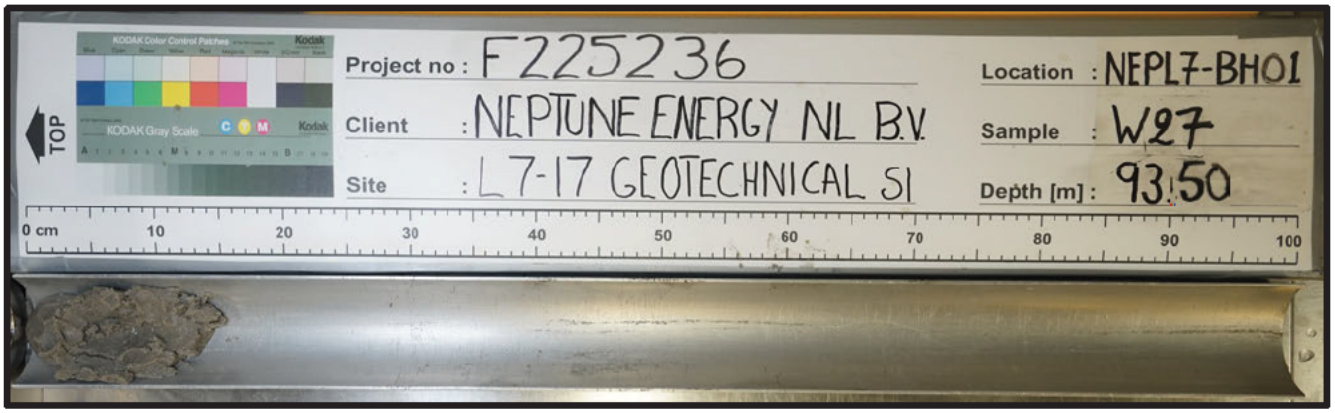
Location	:	NEPL7-BH01
Sample	:	W26
Depth [m BSF]	:	90.00
Note(s)	:	Split

SAMPLE PHOTOGRAPHS

Release Date: 6-3-2023
Drawn with: Photo_Plate_Maker v 6.0
Photo Folder: I Project Workfolders\Neptune L7\Photos\Proc
Presented Plate: Downhole_Portrait
Processed by: DBS
Date: 6-3-2023



Location	:	NEPL7-BH01
Sample	:	W27
Depth [m BSF]	:	93.50
Note(s)	:	Intact

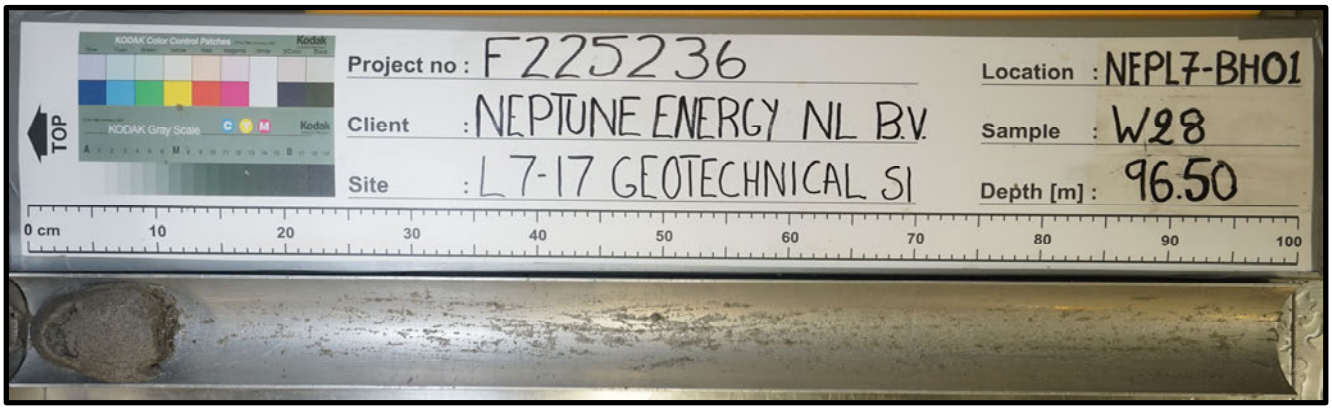


Location	:	NEPL7-BH01
Sample	:	W27
Depth [m BSF]	:	93.50
Note(s)	:	Split

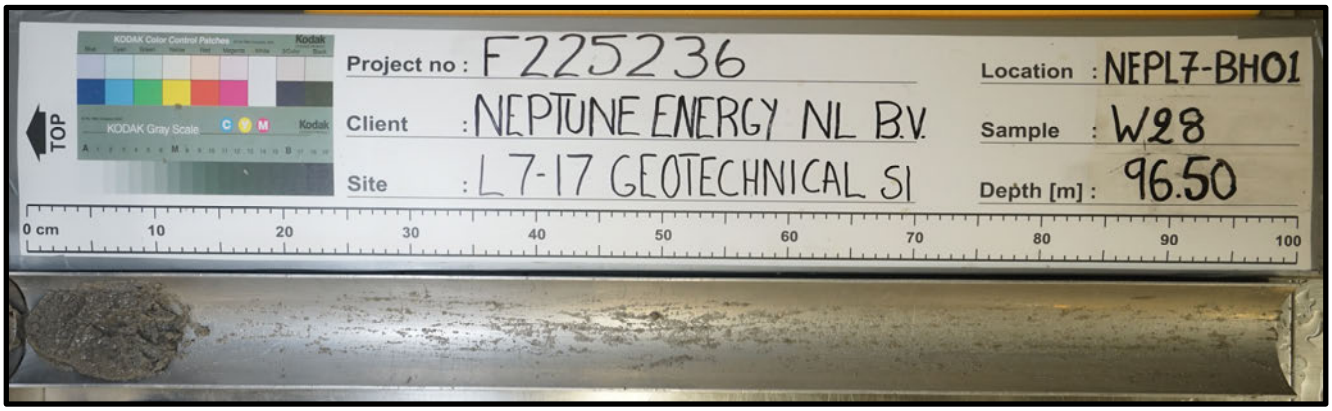
SAMPLE PHOTOGRAPHS

Release Date: 6-3-2023
Drawn with: Photo_Plate_Maker v 6.0
Photo Folder: I Project Workfolders\Neptune L7\Photos\Proc
Presented Plate: Downhole_Portrait
Processed by: DBS
Date: 6-3-2023

Release Date: 6-3-2023
Drawn with: Photo_Plate_Maker v 6.0
Photo Folder: I Project Workfolders\Neptune L7\Photos\Proc
Presented Plate: Downhole_Portrait
Processed by: DBS
Date: 6-3-2023



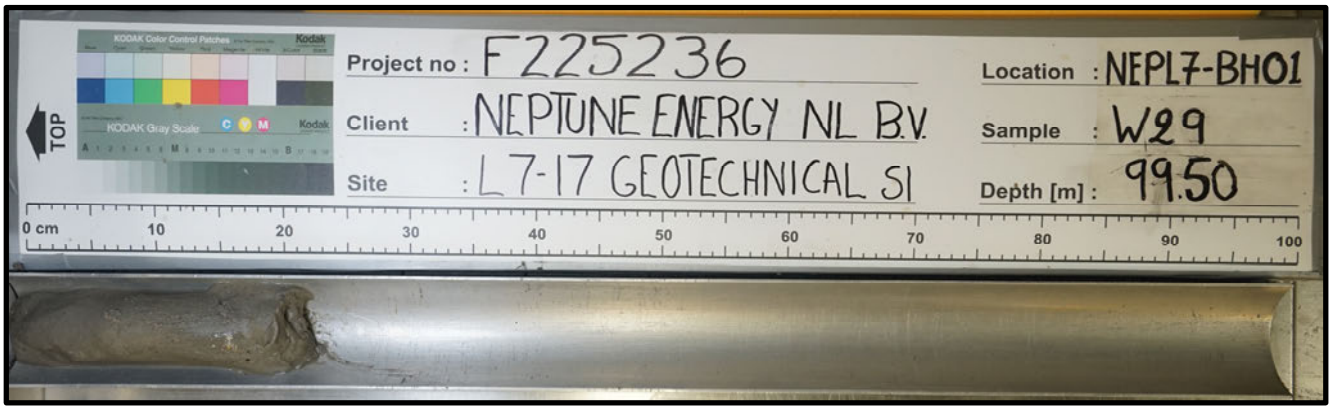
Location	:	NEPL7-BH01
Sample	:	W28
Depth [m BSF]	:	96.50
Note(s)	:	Intact



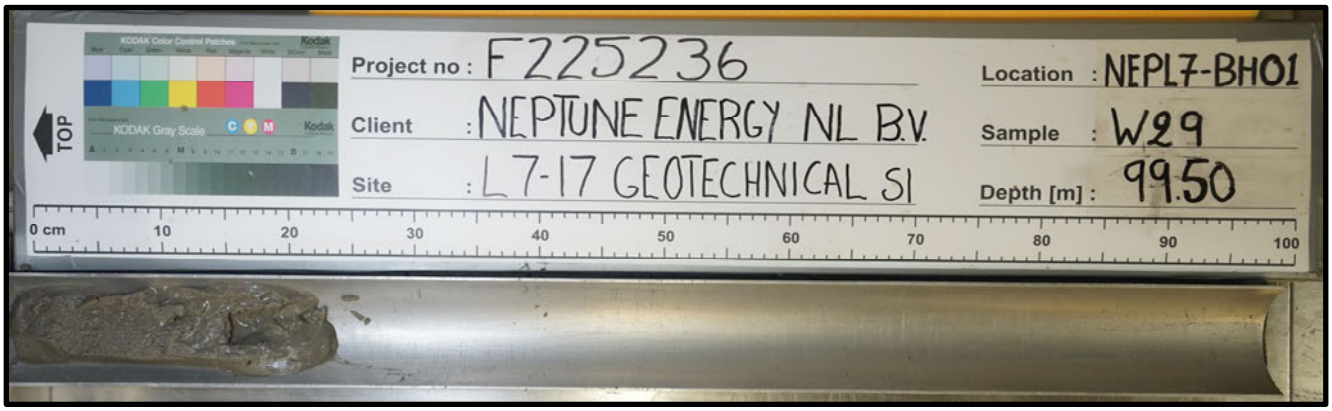
Location	:	NEPL7-BH01
Sample	:	W28
Depth [m BSF]	:	96.50
Note(s)	:	Split

SAMPLE PHOTOGRAPHS

Release Date: 6-3-2023
Drawn with: Photo_Plate_Maker v 6.0
Photo Folder: I Project Workfolders\Neptune L7\Photos\Proc
Presented Plate: Downhole_Portrait
Processed by: DBS
Date: 6-3-2023



Location	:	NEPL7-BH01
Sample	:	W29
Depth [m BSF]	:	99.50
Note(s)	:	Intact



Location	:	NEPL7-BH01
Sample	:	W29
Depth [m BSF]	:	99.50
Note(s)	:	Split

SAMPLE PHOTOGRAPHS

Contents Appendix D.4: Results of Consolidated Drained Triaxial Tests

List of Plates

Title	Plate
Results of Consolidated Drained Triaxial Tests	D.4-1 to D.4-20

FUGRO	**ISOTROPICALLY CONSOLIDATED DRAINED TRIAXIAL COMPRESSION TEST : SINGLE SPECIMEN with measurement of volume change**		Project No.	F225236
			Borehole No.	NEPL7-BH01
			Sample No.	04BAGA
Project Name	Neptune L7 -17		Sample Type	B
Specimen Description	Brown grey silty fine to medium SAND		Sample preparation	RECONSTITUTED
Test Method	Specimen set-up	ISO 17892-9:2018, Par.6.2	Sample orientation	n/a
	Saturation	ISO 17892-9:2018, Par.6.3		
	Consolidation-Isotropic	ISO 17892-9:2018, Par.6.4	Drainage conditions	Both ends
	Shearing	ISO 17892-9:2018, Par.6.7		
Test type	CD	assumed ☑ measured ☐	Specific gravity	2.65
			Date started	25/04/2023

INITIAL CONDITIONS

Sample depth	[m]	2.00		
Specimen depth	[mm]	n/a		
Diameter	[mm]	50.94		
Length	[mm]	100.00		
Moisture content	[%]	10.0		
Bulk density	[Mg/m^3]	1.89		
Dry density	[Mg/m^3]	1.71		
Voids ratio		0.55		
Degree of saturation	[%]	49		

SATURATION STAGE

Final cell pressure	[kPa]	325		
Final pore pressure	[kPa]	313		
Pres. Increments/Differential pres.	[kPa]	50;100 / 10-8		
B value achieved	[%]	99		
Duration	[days]	1		

CONSOLIDATION STAGE

Cell pressure	[kPa]	325		
Back pressure	[kPa]	300		
Effective pressure	[kPa]	25		
Final pore pressure	[kPa]	300		
Final pore pressure dissipation	[%]	100		
Duration	[days]	1		

COMPRESSION STAGE

Cell Pressure	[kPa]	325		
Initial pore pressure	[kPa]	300		
Initial effective stress	[kPa]	25		
Rate of strain	[mm/min]	0.0220		
Duration	[days]	1		

Checked by: 5.1.2.e Date: 27/04/2023 Approved by: 5.1.2.e Date: 27/04/2023

Remarks

Undercompaction & CO_2 saturation applied
Remoulded to 1.71 Mg/m³ dry density

Figure Number

Sheet Number 1/4

FUGRO	**ISOTROPICALLY CONSOLIDATED DRAINED TRIAXIAL COMPRESSION TEST : SINGLE SPECIMEN with measurement of volume change**	Project No.	F225236
		Borehole No.	NEPL7-BH01
		Sample No.	04BAGA
Project Name	Neptune L7 -17	Test type	CD

Conditions at failure

Assumed failure criterion		Maximum deviator stress		
Pore pressure	[kPa]	300		
Minor effective principal stress	[kPa]	25		
Deviator stress	[kPa]	138		
Major effective principal stress	[kPa]	163		
Effective principal stress ratio		6.5		
Axial strain	[%]	1.8		
Volumetric strain	[%]	-1.3		
Corrections to dev. stress PaperSd/Memb	[kPa]	0.0 / 0.6		
Total Duration	[days]	3		

EFFECTIVE STRESS PARAMETERS

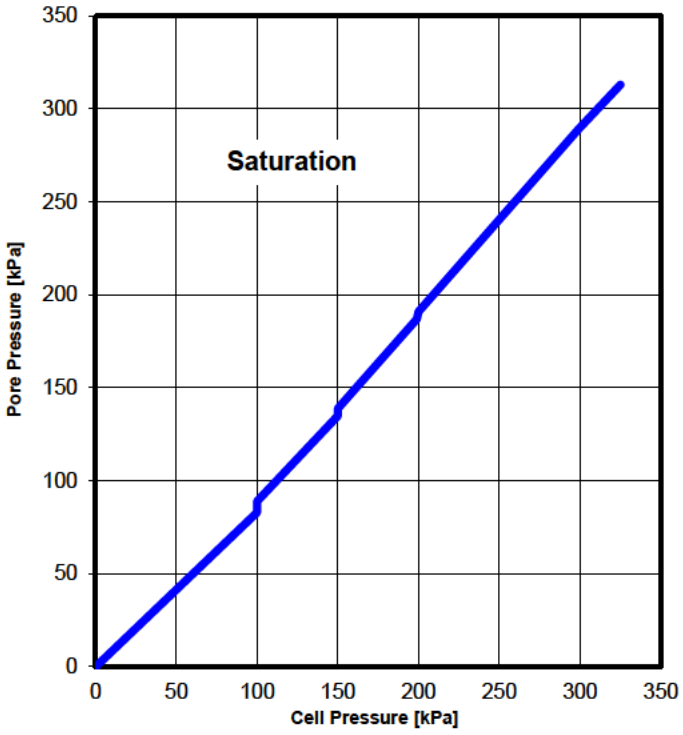
Cohesion c'	[kPa]	n/a
Angle of shearing resistance ϕ'	degrees	n/a

FINAL CONDITIONS (Calculated)

Moisture content	[%]	23.5		
Bulk density	[Mg/m^3]	2.02		

OTHER PARAMETERS

Secant modulus E' 50	[MPa]	32.9		
Strain at ε_{50}	[%]	0.21		



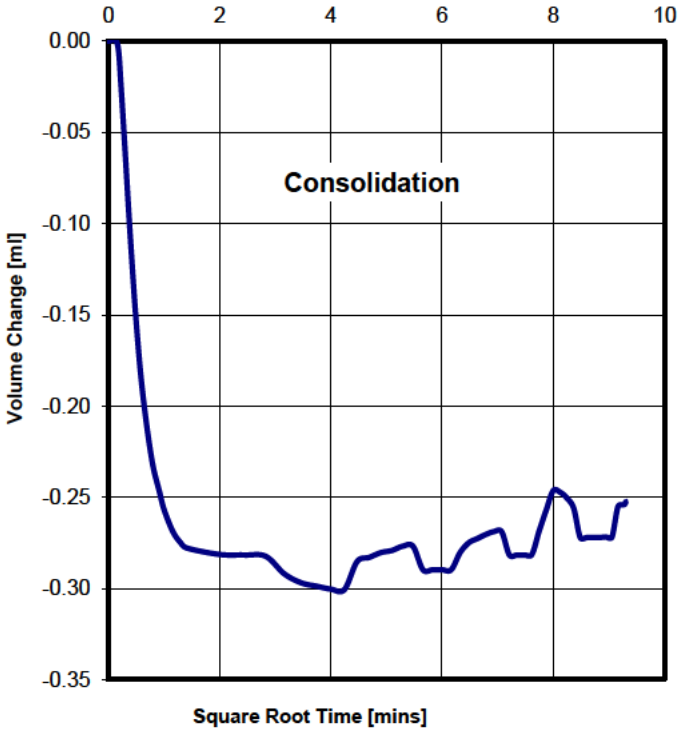
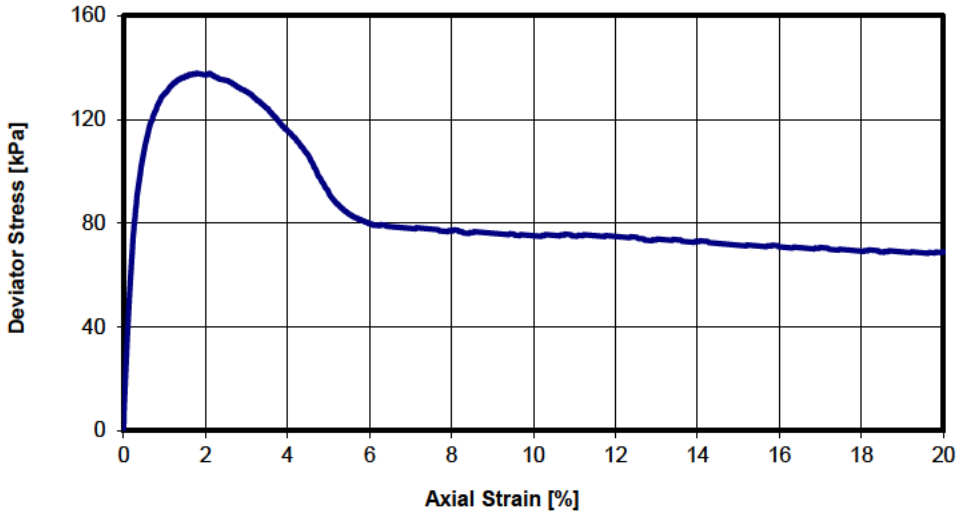
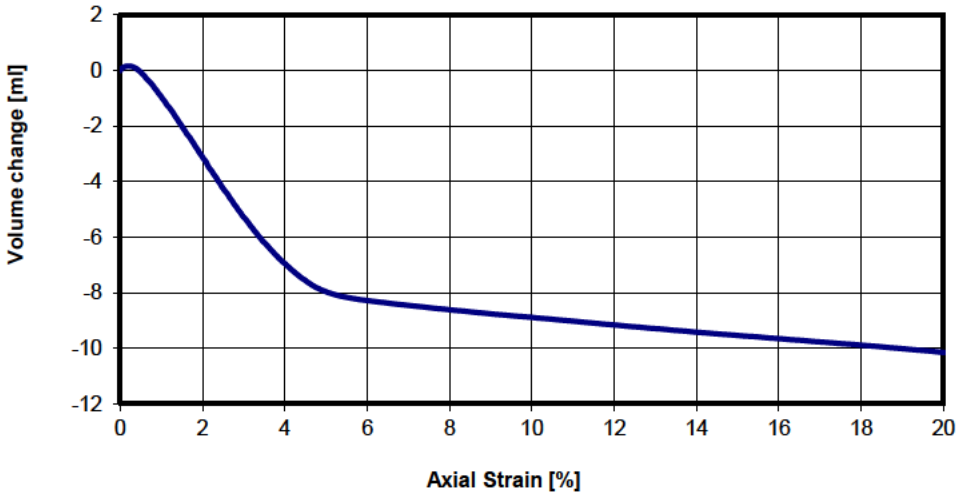


Figure Number	Sheet Number
	2/4

	ISOTROPICALLY CONSOLIDATED DRAINED TRIAXIAL COMPRESSION TEST : SINGLE SPECIMEN with measurement of volume change	Project No.	F225236
		Borehole No.	NEPL7-BH01
		Sample No.	04BAGA
Project Name	Neptune L7 -17	Test type	CD





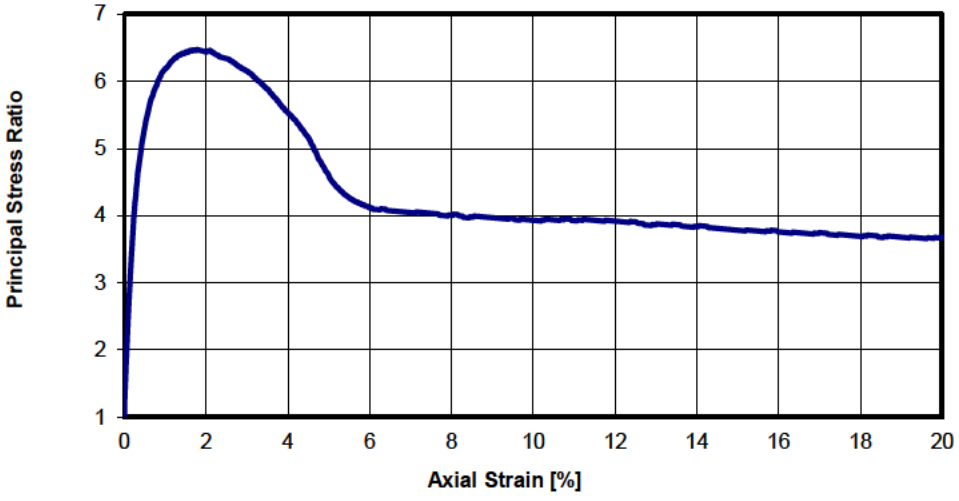
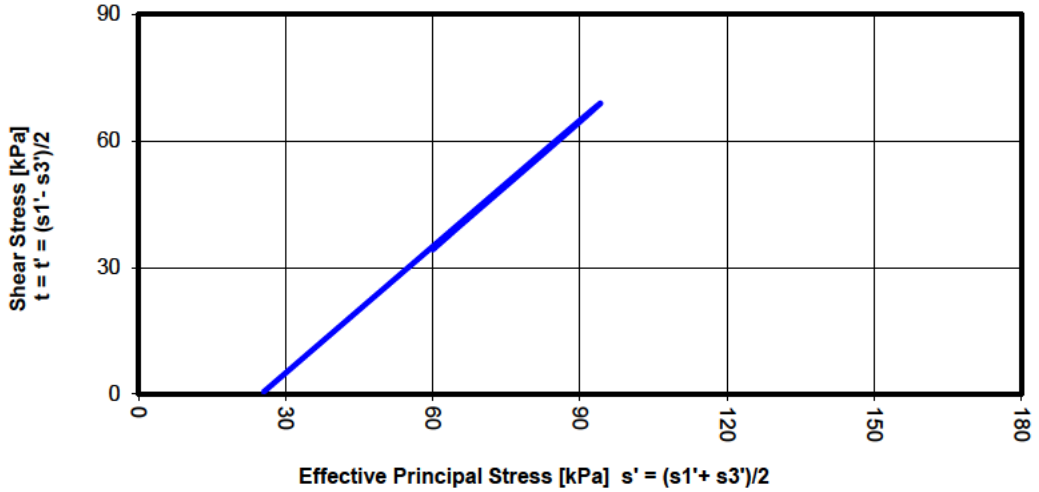
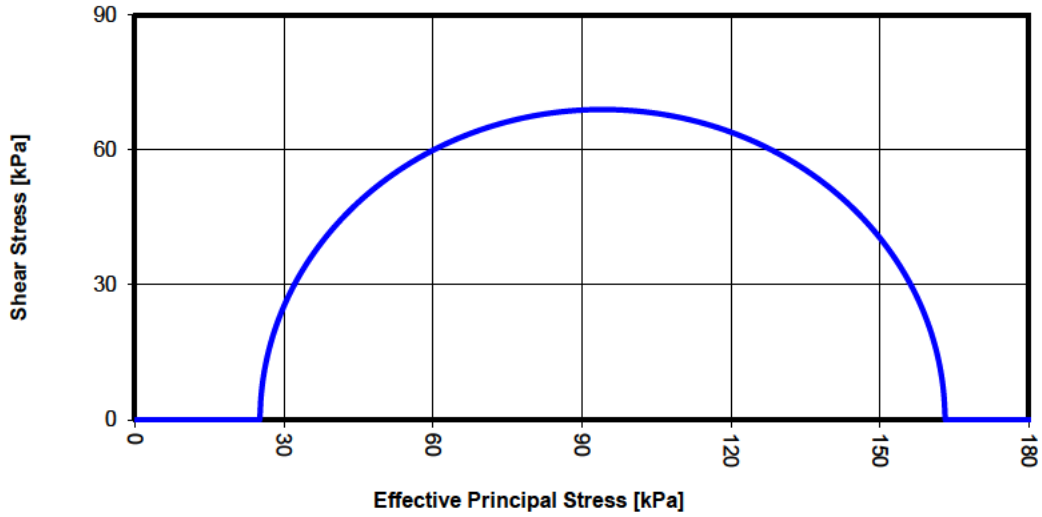


Figure Number	Sheet Number
	3/4

FUGRO	**ISOTROPICALLY CONSOLIDATED DRAINED TRIAXIAL COMPRESSION TEST : SINGLE SPECIMEN with measurement of volume change**	Project No.	F225236
		Borehole No.	NEPL7-BH01
		Sample No.	04BAGA
Project Name	Neptune L7 -17	Test type	CD





Mode of failure:

Brittle

Figure Number	Sheet Number
	4/4

FUGRO	**ISOTROPICALLY CONSOLIDATED DRAINED TRIAXIAL COMPRESSION TEST : SINGLE SPECIMEN with measurement of volume change**		Project No.	F225236
			Borehole No.	NEPL7-BH01
			Sample No.	05BAGB
Project Name	Neptune L7 -17		Sample Type	B
Specimen Description	Grey slightly clayey sandy SILT		Sample preparation	RECONSTITUTED
Test Method	Specimen set-up	ISO 17892-9:2018, Par.6.2	Sample orientation	n/a
	Saturation	ISO 17892-9:2018, Par.6.3		
	Consolidation-Isotropic	ISO 17892-9:2018, Par.6.4	Drainage conditions	Both ends
	Shearing	ISO 17892-9:2018, Par.6.7		
Test type	CD	assumed ☑ measured ☐	Specific gravity	2.65
			Date started	27/04/2023

INITIAL CONDITIONS

Sample depth	[m]	31.25		
Specimen depth	[mm]	n/a		
Diameter	[mm]	50.94		
Length	[mm]	100.00		
Moisture content	[%]	10.4		
Bulk density	[Mg/m^3]	1.77		
Dry density	[Mg/m^3]	1.61		
Voids ratio		0.65		
Degree of saturation	[%]	42		

SATURATION STAGE

Final cell pressure	[kPa]	535		
Final pore pressure	[kPa]	522		
Pres. Increments/Differential pres.	[kPa]	50;100 / 10-6		
B value achieved	[%]	99		
Duration	[days]	1		

CONSOLIDATION STAGE

Cell pressure	[kPa]	535		
Back pressure	[kPa]	300		
Effective pressure	[kPa]	235		
Final pore pressure	[kPa]	300		
Final pore pressure dissipation	[%]	100		
Duration	[days]	1		

COMPRESSION STAGE

Cell Pressure	[kPa]	535		
Initial pore pressure	[kPa]	300		
Initial effective stress	[kPa]	235		
Rate of strain	[mm/min]	0.0220		
Duration	[days]	1		

Checked by: 5.1.2.e Date: 03/05/2023 Approved by: 5.1.2.e Date: 04/05/2023

Remarks

Undercompaction & CO_2 saturation applied
Remoulded to 1.61 Mg/m³ dry density

Figure Number

Sheet Number 1/4

FUGRO	**ISOTROPICALLY CONSOLIDATED DRAINED TRIAXIAL COMPRESSION TEST : SINGLE SPECIMEN with measurement of volume change**	Project No.	F225236
		Borehole No.	NEPL7-BH01
		Sample No.	05BAGB
Project Name	Neptune L7 -17	Test type	CD

Conditions at failure

Assumed failure criterion		Maximum deviator stress		
Pore pressure	[kPa]	300		
Minor effective principal stress	[kPa]	234		
Deviator stress	[kPa]	532		
Major effective principal stress	[kPa]	767		
Effective principal stress ratio		3.3		
Axial strain	[%]	19.1		
Volumetric strain	[%]	4.5		
Corrections to dev. stress PaperSd/Memb	[kPa]	0.0 / 9.4		
Total Duration	[days]	3		

EFFECTIVE STRESS PARAMETERS

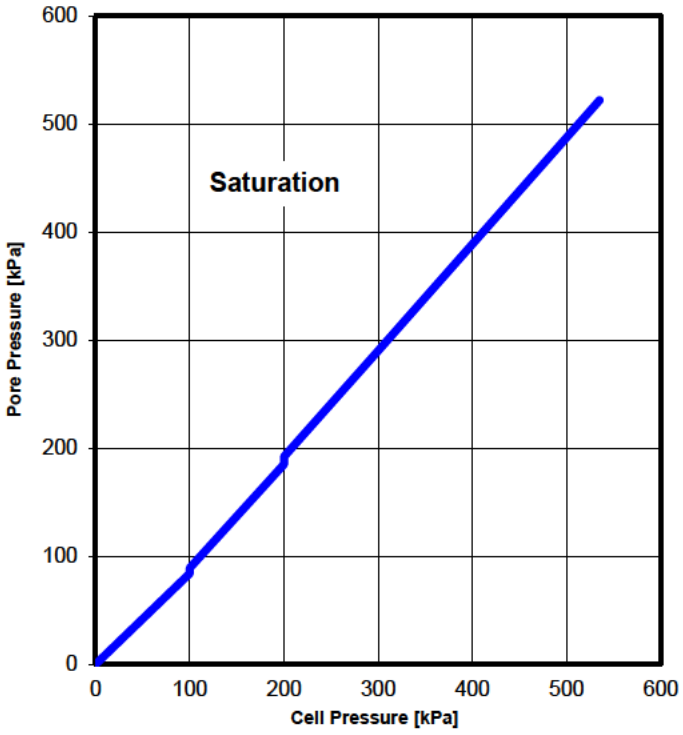
Cohesion c'	[kPa]	n/a
Angle of shearing resistance ϕ'	degrees	n/a

FINAL CONDITIONS (Calculated)

Moisture content	[%]	18.2		
Bulk density	[Mg/m^3]	2.11		

OTHER PARAMETERS

Secant modulus E' 50	[MPa]	7.5		
Strain at ε_{50}	[%]	3.55		



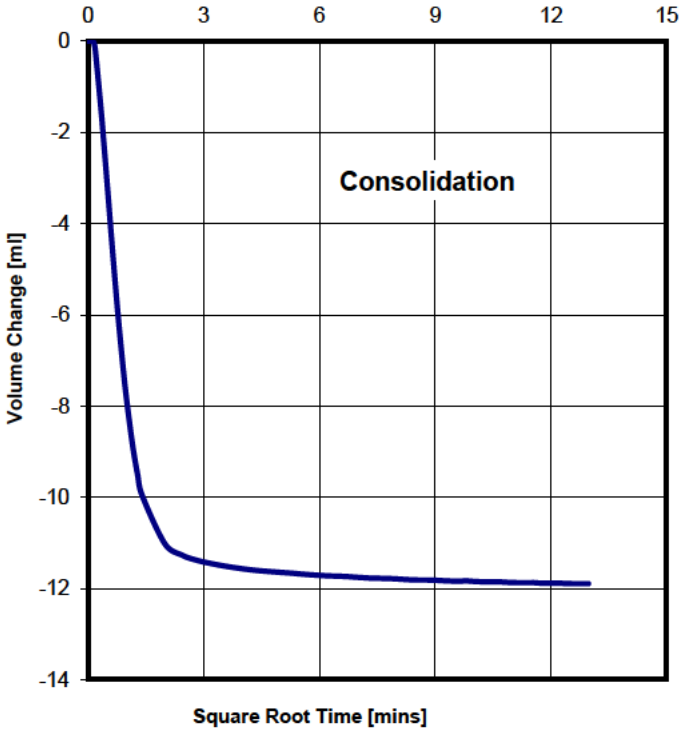


Figure Number	Sheet Number
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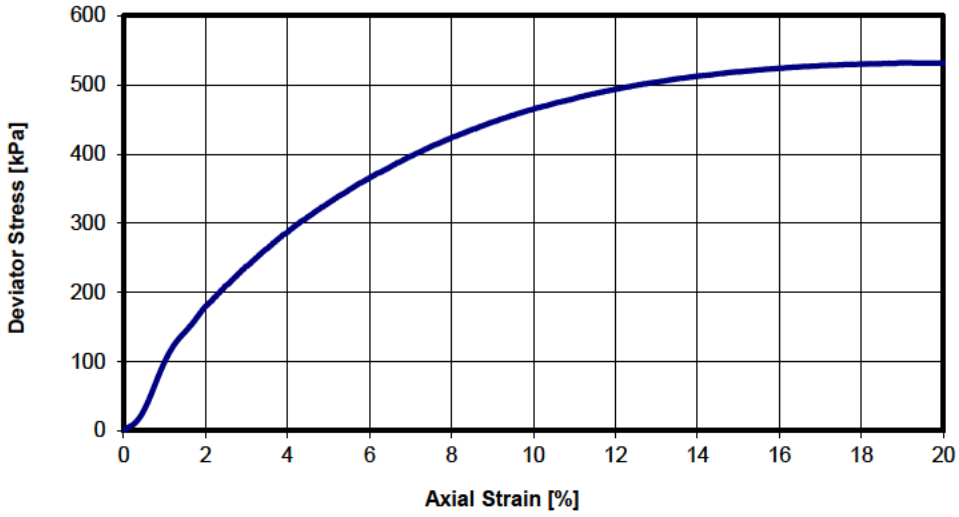
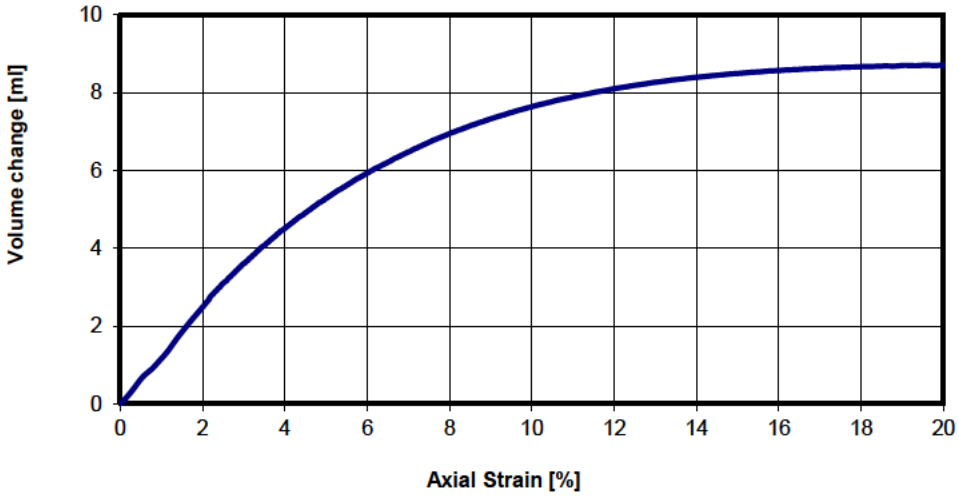
FUGRO	**ISOTROPICALLY CONSOLIDATED DRAINED TRIAXIAL COMPRESSION TEST : SINGLE SPECIMEN with measurement of volume change**	Project No.	F225236
		Borehole No.	NEPL7-BH01
		Sample No.	05BAGB
Project Name	Neptune L7 -17	Test type	CD





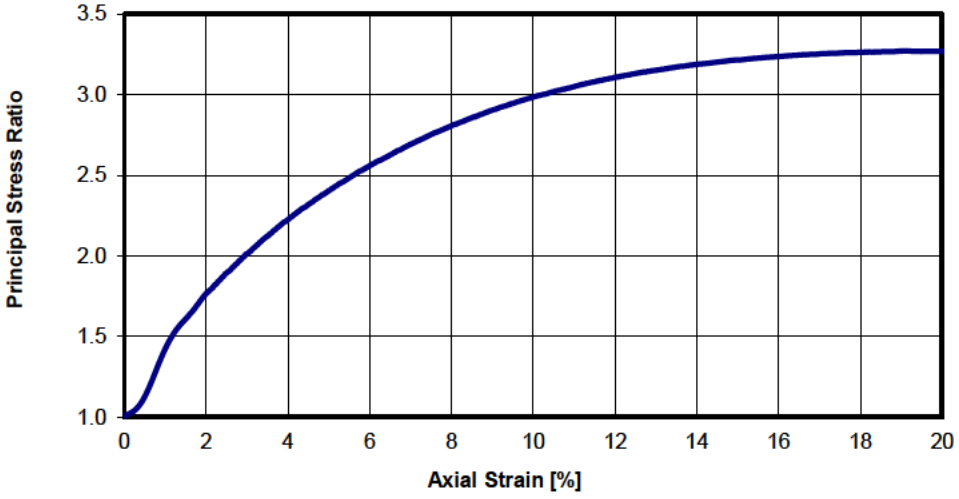
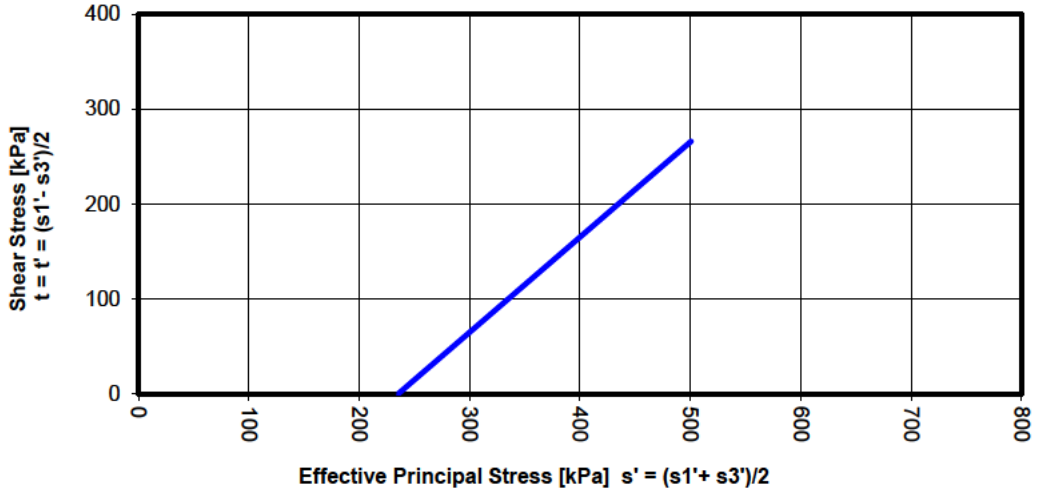
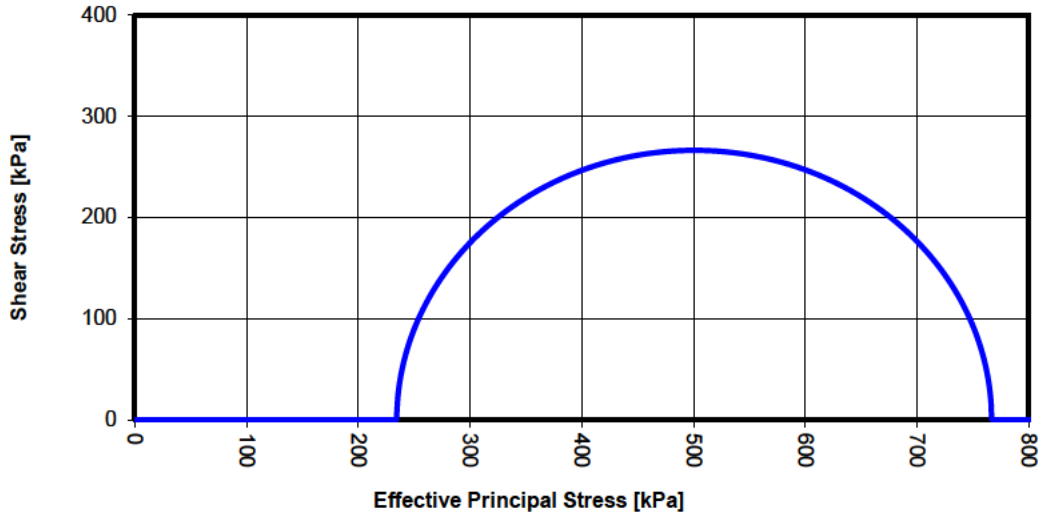


Figure Number	Sheet Number
	3/4

FUGRO	**ISOTROPICALLY CONSOLIDATED DRAINED TRIAXIAL COMPRESSION TEST : SINGLE SPECIMEN with measurement of volume change**	Project No.	F225236
		Borehole No.	NEPL7-BH01
		Sample No.	05BAGB
Project Name	Neptune L7 -17	Test type	CD





Mode of failure:

Plastic

Figure Number	Sheet Number
	4/4

FUGRO	**ISOTROPICALLY CONSOLIDATED DRAINED TRIAXIAL COMPRESSION TEST : SINGLE SPECIMEN with measurement of volume change**		Project No.	F225236
			Borehole No.	NEPL7-BH01
			Sample No.	10BAGA
Project Name	Neptune L7 -17		Sample Type	B
Specimen Description	Grey slightly silty fine to medium SAND		Sample preparation	RECONSTITUTED
Test Method	Specimen set-up	ISO 17892-9:2018, Par.6.2	Sample orientation	n/a
	Saturation	ISO 17892-9:2018, Par.6.3		
	Consolidation-Isotropic	ISO 17892-9:2018, Par.6.4	Drainage conditions	Both ends
	Shearing	ISO 17892-9:2018, Par.6.7		
Test type	CD	assumed ☑ measured ☐	Specific gravity	2.65
			Date started	20/04/2023

INITIAL CONDITIONS

Sample depth	[m]	42.50		
Specimen depth	[mm]	n/a		
Diameter	[mm]	50.94		
Length	[mm]	100.00		
Moisture content	[%]	10.0		
Bulk density	[Mg/m^3]	1.74		
Dry density	[Mg/m^3]	1.58		
Voids ratio		0.67		
Degree of saturation	[%]	40		

SATURATION STAGE

Final cell pressure	[kPa]	565		
Final pore pressure	[kPa]	548		
Pres. Increments/Differential pres.	[kPa]	50;100 / 10-6		
B value achieved	[%]	97		
Duration	[days]	1		

CONSOLIDATION STAGE

Cell pressure	[kPa]	565		
Back pressure	[kPa]	300		
Effective pressure	[kPa]	265		
Final pore pressure	[kPa]	300		
Final pore pressure dissipation	[%]	100		
Duration	[days]	1		

COMPRESSION STAGE

Cell Pressure	[kPa]	565		
Initial pore pressure	[kPa]	300		
Initial effective stress	[kPa]	265		
Rate of strain	[mm/min]	0.0220		
Duration	[days]	1		

Checked by: 5.1.2.e Date: 26/04/2023 Approved by: 5.1.2.e Date: 27/04/2023

Remarks

Undercompaction & CO_2 saturation applied Remoulded to 1.58 Mg/m³ dry density	Figure Number	Sheet Number 1/4

FUGRO	**ISOTROPICALLY CONSOLIDATED DRAINED TRIAXIAL COMPRESSION TEST : SINGLE SPECIMEN with measurement of volume change**	Project No.	F225236
		Borehole No.	NEPL7-BH01
		Sample No.	10BAGA
Project Name	Neptune L7 -17	Test type	CD

Conditions at failure

Assumed failure criterion		Maximum deviator stress		
Pore pressure	[kPa]	300		
Minor effective principal stress	[kPa]	265		
Deviator stress	[kPa]	749		
Major effective principal stress	[kPa]	1014		
Effective principal stress ratio		3.8		
Axial strain	[%]	3.8		
Volumetric strain	[%]	-0.4		
Corrections to dev. stress PaperSd/Memb	[kPa]	0.0 / 1.7		
Total Duration	[days]	3		

EFFECTIVE STRESS PARAMETERS

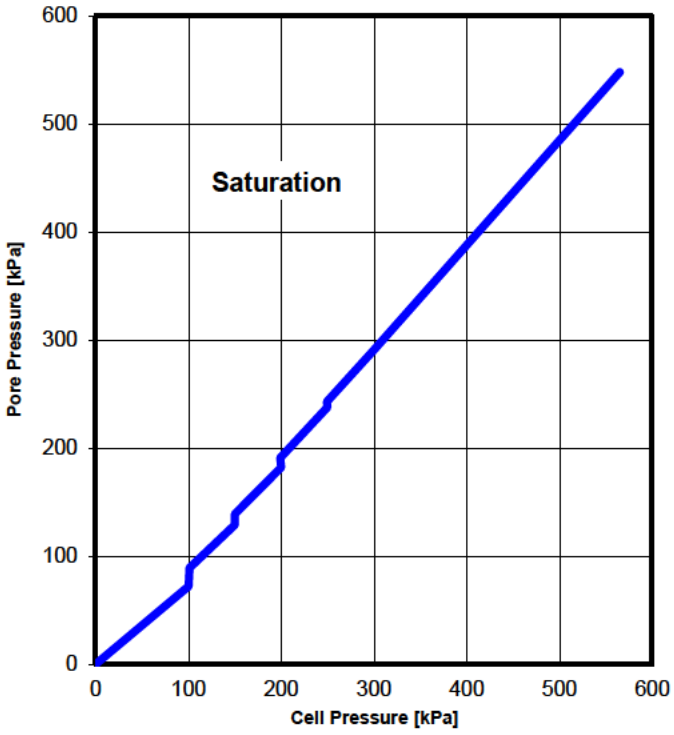
Cohesion c'	[kPa]	n/a
Angle of shearing resistance ϕ'	degrees	n/a

FINAL CONDITIONS (Calculated)

Moisture content	[%]	26.1		
Bulk density	[Mg/m^3]	1.98		

OTHER PARAMETERS

Secant modulus E' 50	[MPa]	90.5		
Strain at ε_{50}	[%]	0.41		



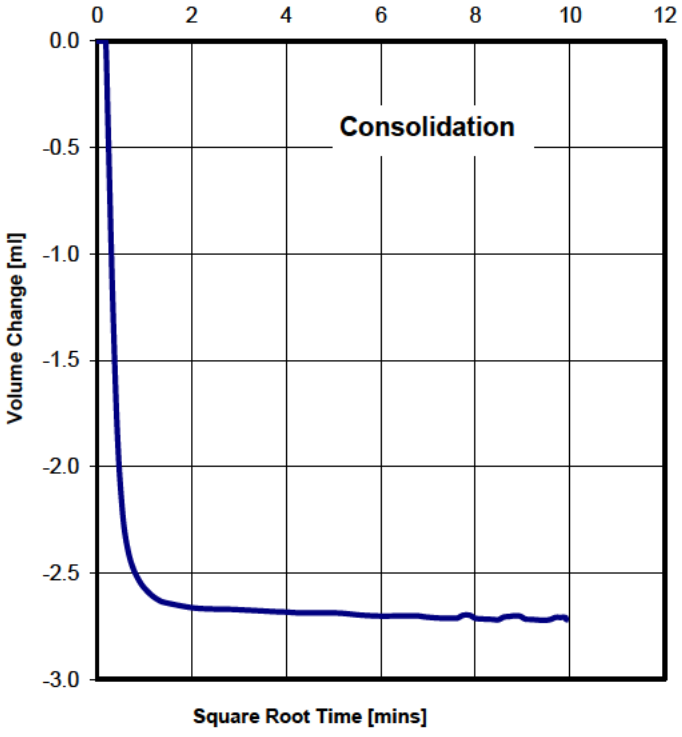


Figure Number	Sheet Number
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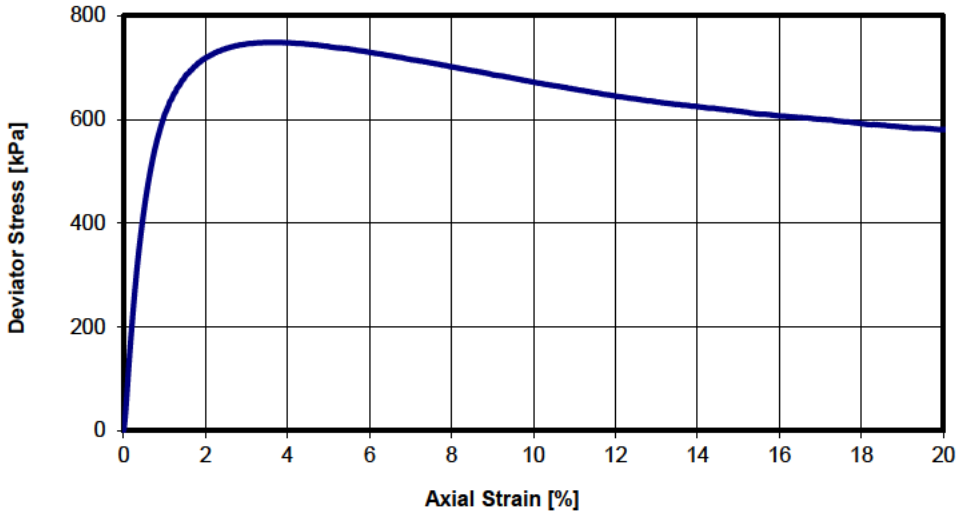
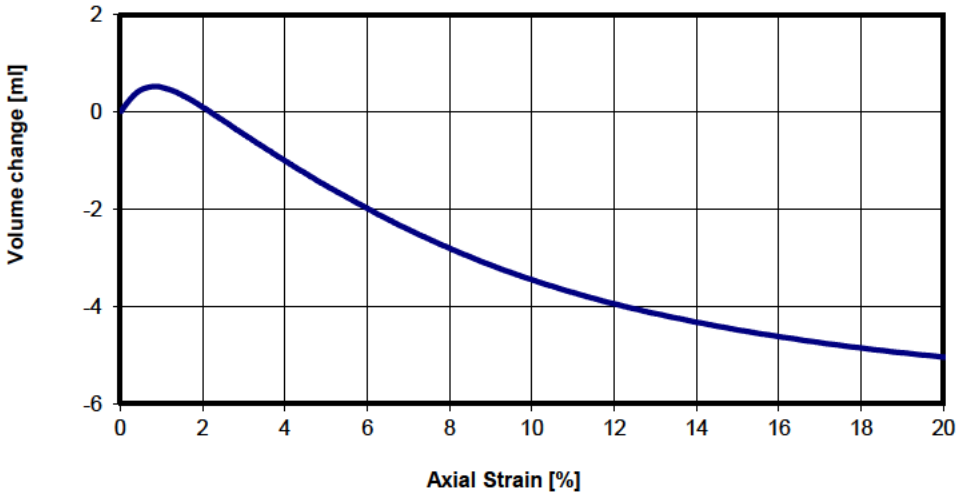
	ISOTROPICALLY CONSOLIDATED DRAINED TRIAXIAL COMPRESSION TEST : SINGLE SPECIMEN with measurement of volume change	Project No.	F225236
		Borehole No.	NEPL7-BH01
		Sample No.	10BAGA
Project Name	Neptune L7 -17	Test type	CD





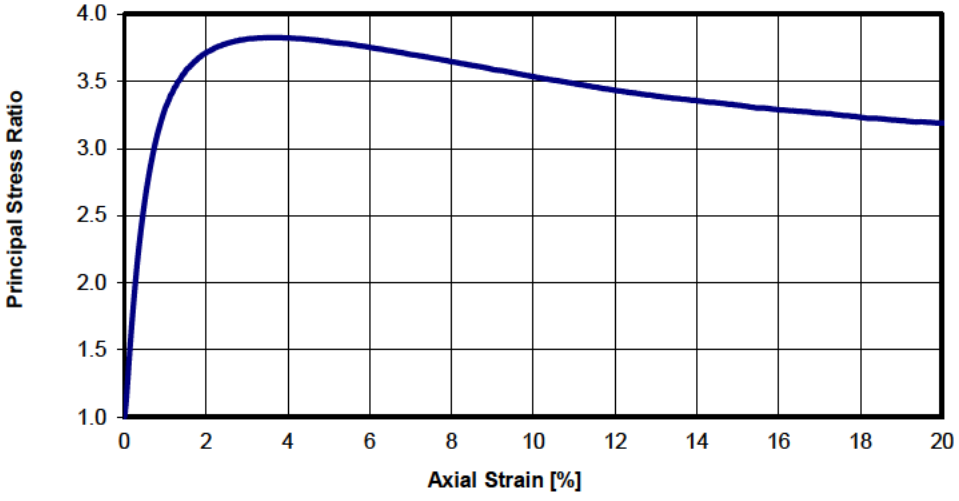
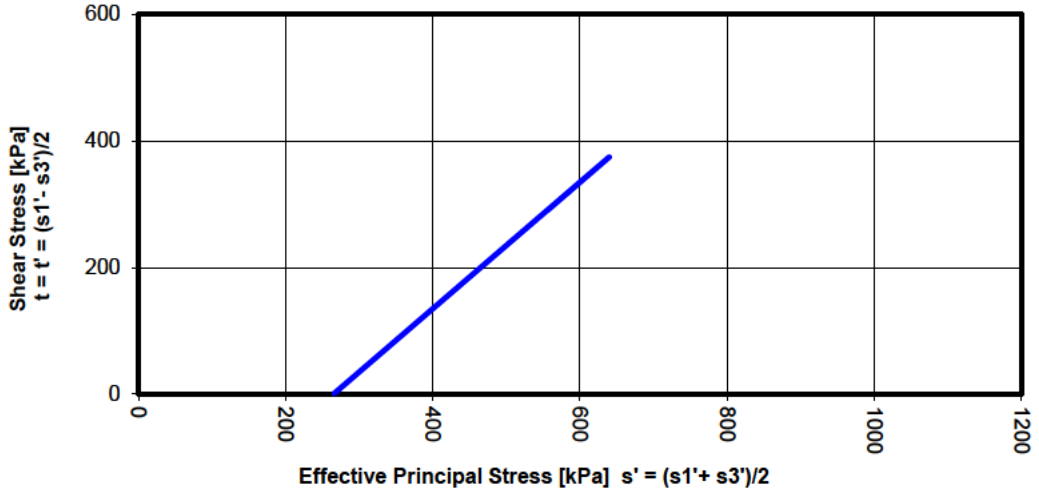
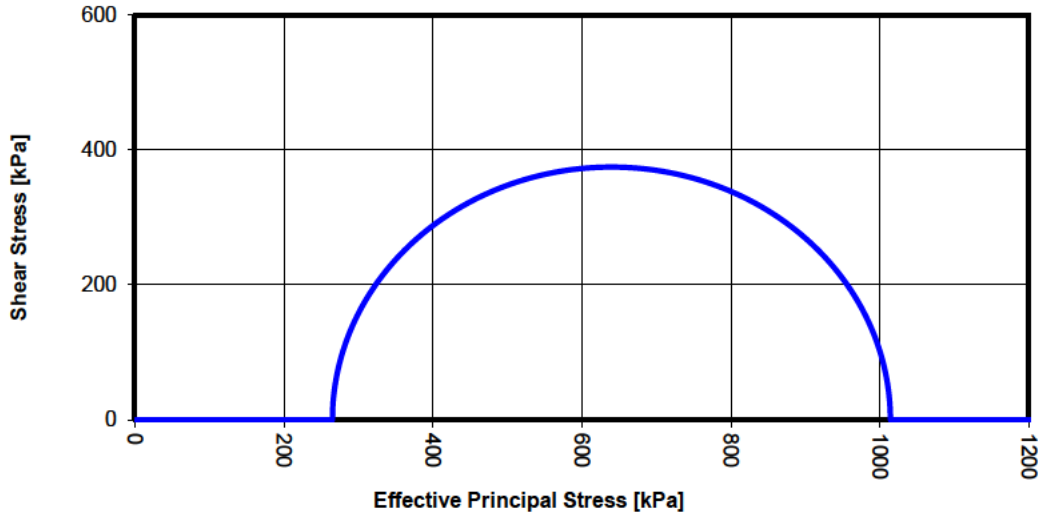


Figure Number	Sheet Number
	3/4

FUGRO	**ISOTROPICALLY CONSOLIDATED DRAINED TRIAXIAL COMPRESSION TEST : SINGLE SPECIMEN with measurement of volume change**	Project No.	F225236
		Borehole No.	NEPL7-BH01
		Sample No.	10BAGA
Project Name	Neptune L7 -17	Test type	CD





Mode of failure:

Plastic

Figure Number	Sheet Number
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FUGRO	**ISOTROPICALLY CONSOLIDATED DRAINED TRIAXIAL COMPRESSION TEST : SINGLE SPECIMEN with measurement of volume change**		Project No.	F225236
			Borehole No.	NEPL7-BH01
			Sample No.	13BAGA
Project Name	Neptune L7 -17		Sample Type	B
Specimen Description	Grey slightly silty fine SAND		Sample preparation	RECONSTITUTED
Test Method	Specimen set-up	ISO 17892-9:2018, Par.6.2	Sample orientation	n/a
	Saturation	ISO 17892-9:2018, Par.6.3		
	Consolidation-Isotropic	ISO 17892-9:2018, Par.6.4	Drainage conditions	Both ends
	Shearing	ISO 17892-9:2018, Par.6.7		
Test type	CD	assumed ☑ measured ☐	Specific gravity	2.65
			Date started	27/04/2023

INITIAL CONDITIONS

Sample depth	[m]	47.00		
Specimen depth	[mm]	n/a		
Diameter	[mm]	50.94		
Length	[mm]	100.00		
Moisture content	[%]	10.3		
Bulk density	[Mg/m^3]	1.80		
Dry density	[Mg/m^3]	1.64		
Voids ratio		0.62		
Degree of saturation	[%]	44		

SATURATION STAGE

Final cell pressure	[kPa]	579		
Final pore pressure	[kPa]	563		
Pres. Increments/Differential pres.	[kPa]	50;100 / 10-6		
B value achieved	[%]	97		
Duration	[days]	1		

CONSOLIDATION STAGE

Cell pressure	[kPa]	580		
Back pressure	[kPa]	300		
Effective pressure	[kPa]	280		
Final pore pressure	[kPa]	300		
Final pore pressure dissipation	[%]	100		
Duration	[days]	1		

COMPRESSION STAGE

Cell Pressure	[kPa]	580		
Initial pore pressure	[kPa]	300		
Initial effective stress	[kPa]	280		
Rate of strain	[mm/min]	0.0220		
Duration	[days]	1		

Checked by: 5.1.2.e Date: 03/05/2023 Approved by: 5.1.2.e Date: 04/05/2023

Remarks

Undercompaction & CO_2 saturation applied
Remoulded to 1.64 Mg/m³ dry density

Figure Number

Sheet Number 1/4

FUGRO	**ISOTROPICALLY CONSOLIDATED DRAINED TRIAXIAL COMPRESSION TEST : SINGLE SPECIMEN with measurement of volume change**	Project No.	F225236
		Borehole No.	NEPL7-BH01
		Sample No.	13BAGA
Project Name	Neptune L7 -17	Test type	CD

Conditions at failure

Assumed failure criterion		Maximum deviator stress		
Pore pressure	[kPa]	300		
Minor effective principal stress	[kPa]	280		
Deviator stress	[kPa]	925		
Major effective principal stress	[kPa]	1205		
Effective principal stress ratio		4.3		
Axial strain	[%]	3.5		
Volumetric strain	[%]	-0.4		
Corrections to dev. stress PaperSd/Memb	[kPa]	0.0 / 1.5		
Total Duration	[days]	3		

EFFECTIVE STRESS PARAMETERS

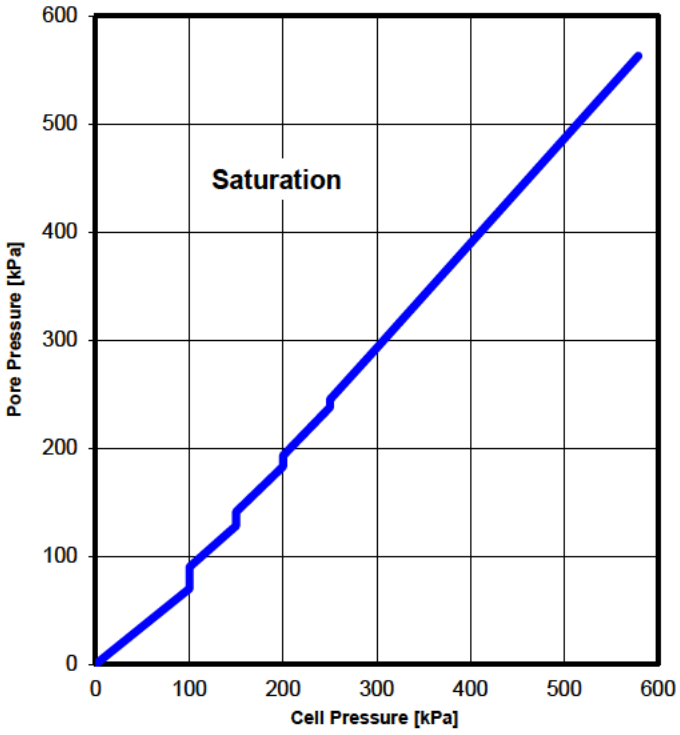
Cohesion c'	[kPa]	n/a
Angle of shearing resistance ϕ'	degrees	n/a

FINAL CONDITIONS (Calculated)

Moisture content	[%]	23.6		
Bulk density	[Mg/m^3]	2.01		

OTHER PARAMETERS

Secant modulus E' 50	[MPa]	94.5		
Strain at ε_{50}	[%]	0.49		



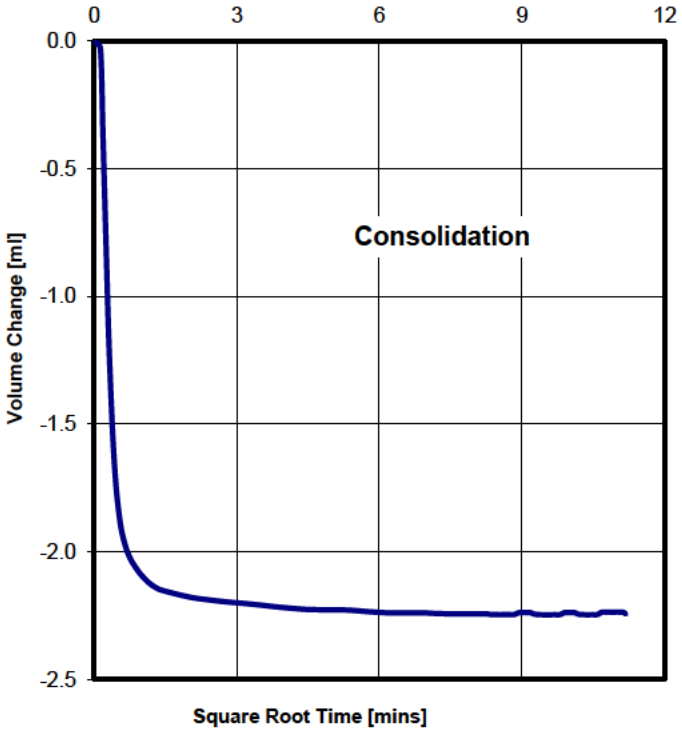


Figure Number	Sheet Number
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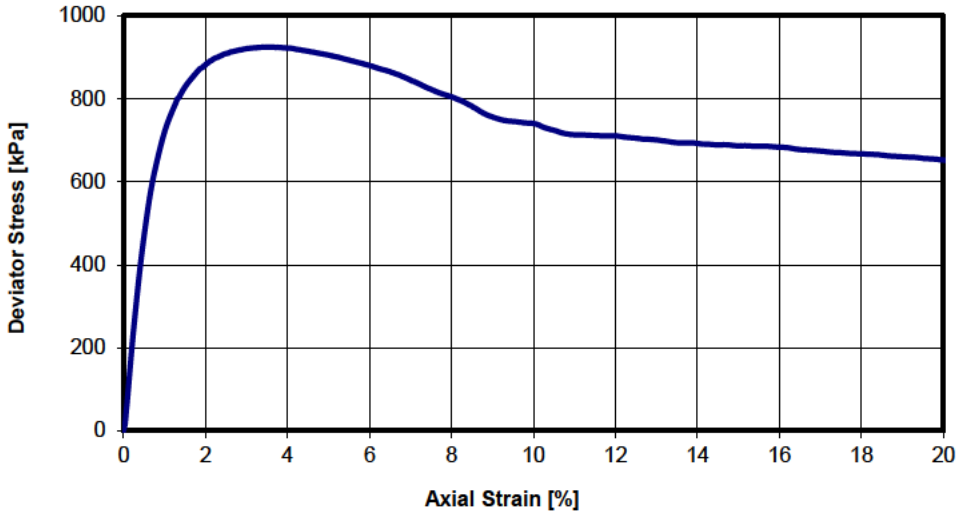
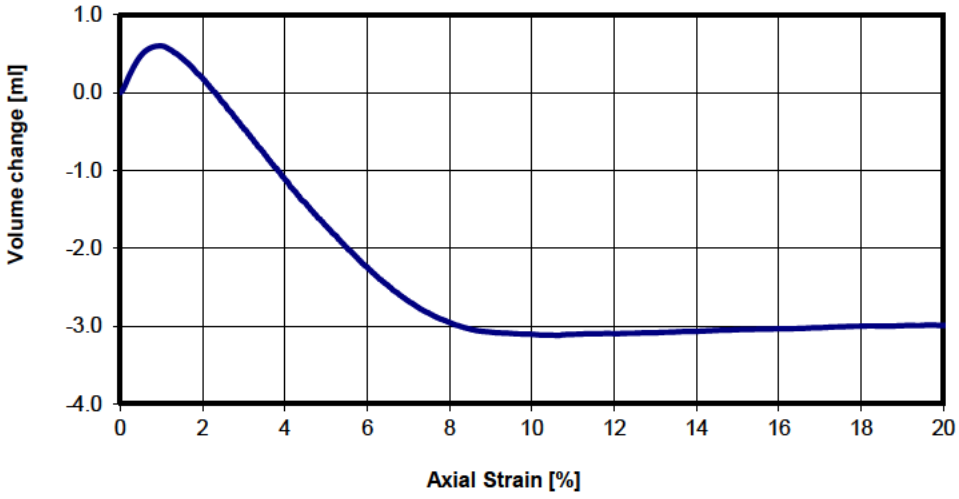
FUGRO	**ISOTROPICALLY CONSOLIDATED DRAINED TRIAXIAL COMPRESSION TEST : SINGLE SPECIMEN with measurement of volume change**	Project No.	F225236
		Borehole No.	NEPL7-BH01
		Sample No.	13BAGA
Project Name	Neptune L7 -17	Test type	CD





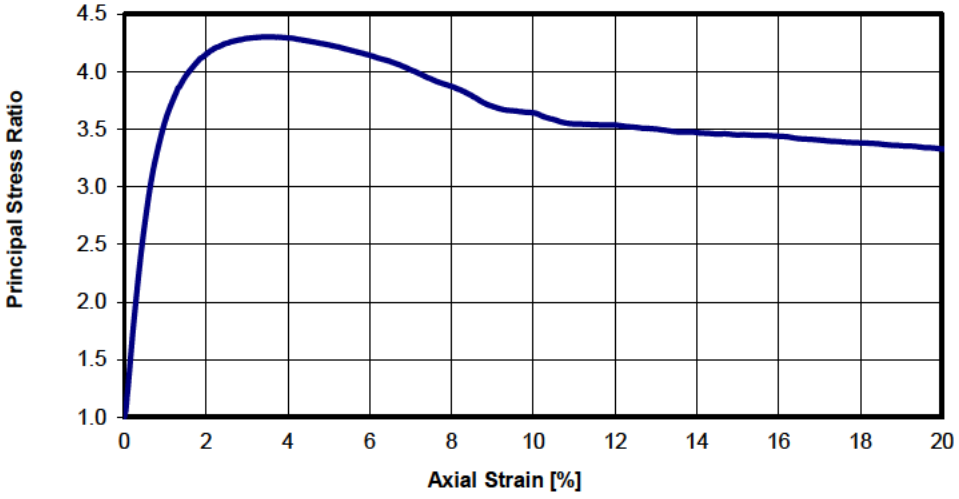


Figure Number	Sheet Number
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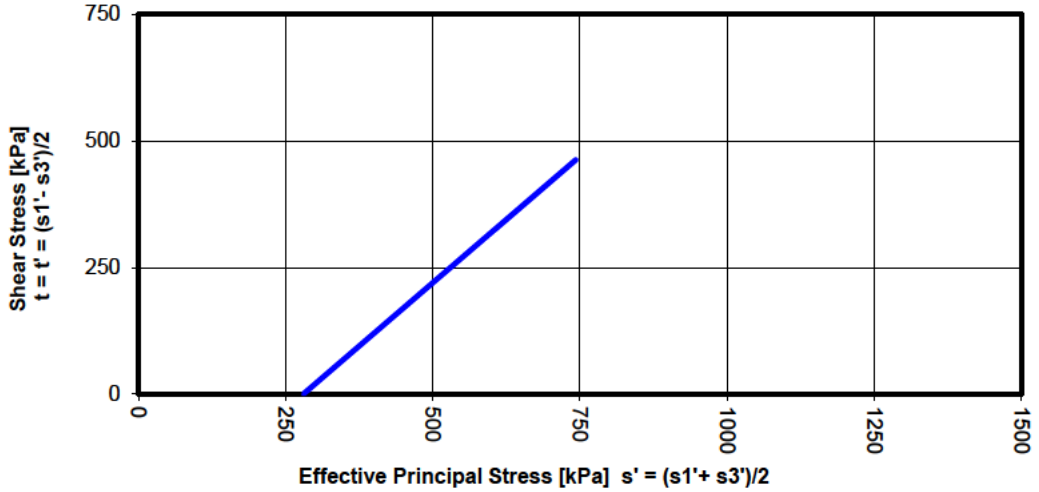
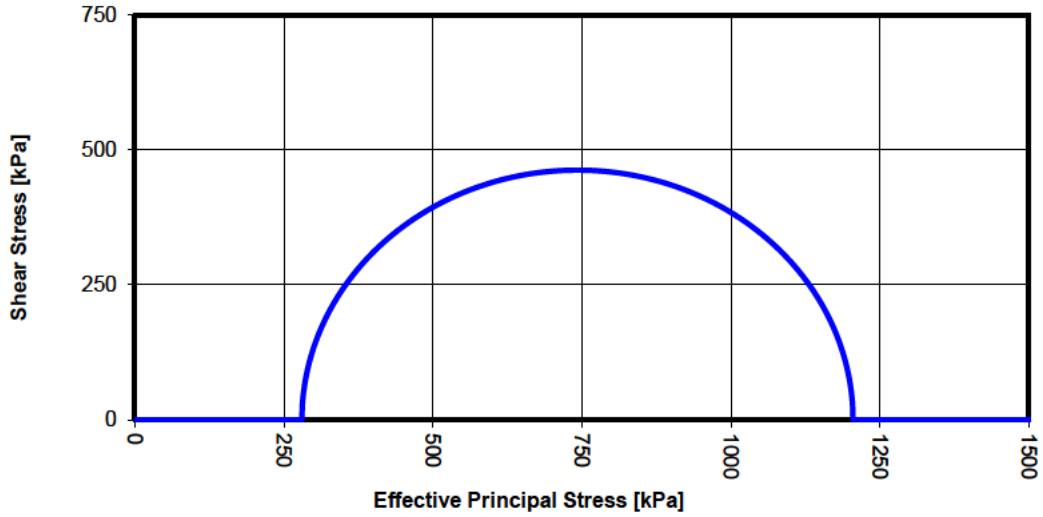
FUGRO	**ISOTROPICALLY CONSOLIDATED DRAINED TRIAXIAL COMPRESSION TEST : SINGLE SPECIMEN with measurement of volume change**	Project No.	F225236
		Borehole No.	NEPL7-BH01
		Sample No.	13BAGA
Project Name	Neptune L7 -17	Test type	CD





Mode of failure:

Brittle

Figure Number	Sheet Number
	4/4

FUGRO	**ISOTROPICALLY CONSOLIDATED DRAINED TRIAXIAL COMPRESSION TEST : SINGLE SPECIMEN with measurement of volume change**		Project No.	F225236
			Borehole No.	NEPL7-BH01
			Sample No.	18BAGA
Project Name	Neptune L7 -17		Sample Type	B
Specimen Description	Grey slightly silty fine to medium SAND		Sample preparation	RECONSTITUTED
Test Method	Specimen set-up	ISO 17892-9:2018, Par.6.2	Sample orientation	n/a
	Saturation	ISO 17892-9:2018, Par.6.3		
	Consolidation-Isotropic	ISO 17892-9:2018, Par.6.4	Drainage conditions	Both ends
	Shearing	ISO 17892-9:2018, Par.6.7		
Test type	CD	assumed ☑ measured ☐	Specific gravity	2.65
			Date started	20/04/2023

INITIAL CONDITIONS

Sample depth	[m]	61.50		
Specimen depth	[mm]	n/a		
Diameter	[mm]	50.94		
Length	[mm]	100.00		
Moisture content	[%]	10.1		
Bulk density	[Mg/m^3]	1.75		
Dry density	[Mg/m^3]	1.59		
Voids ratio		0.67		
Degree of saturation	[%]	40		

SATURATION STAGE

Final cell pressure	[kPa]	645		
Final pore pressure	[kPa]	629		
Pres. Increments/Differential pres.	[kPa]	50;100 / 10-6		
B value achieved	[%]	98		
Duration	[days]	1		

CONSOLIDATION STAGE

Cell pressure	[kPa]	645		
Back pressure	[kPa]	300		
Effective pressure	[kPa]	345		
Final pore pressure	[kPa]	300		
Final pore pressure dissipation	[%]	100		
Duration	[days]	1		

COMPRESSION STAGE

Cell Pressure	[kPa]	645		
Initial pore pressure	[kPa]	300		
Initial effective stress	[kPa]	345		
Rate of strain	[mm/min]	0.0220		
Duration	[days]	1		

Checked by: 5.1.2.e Date: 26/04/2023 Approved by: 5.1.2.e Date: 27/04/2023

Remarks

Undercompaction & CO_2 saturation applied Remoulded to 1.59 Mg/m³ dry density	Figure Number	Sheet Number 1/4

FUGRO	**ISOTROPICALLY CONSOLIDATED DRAINED TRIAXIAL COMPRESSION TEST : SINGLE SPECIMEN with measurement of volume change**	Project No.	F225236
		Borehole No.	NEPL7-BH01
		Sample No.	18BAGA
Project Name	Neptune L7 -17	Test type	CD

Conditions at failure

Assumed failure criterion		Maximum deviator stress		
Pore pressure	[kPa]	300		
Minor effective principal stress	[kPa]	345		
Deviator stress	[kPa]	965		
Major effective principal stress	[kPa]	1310		
Effective principal stress ratio		3.8		
Axial strain	[%]	6.3		
Volumetric strain	[%]	0.1		
Corrections to dev. stress PaperSd/Memb	[kPa]	0.0 / 2.9		
Total Duration	[days]	3		

EFFECTIVE STRESS PARAMETERS

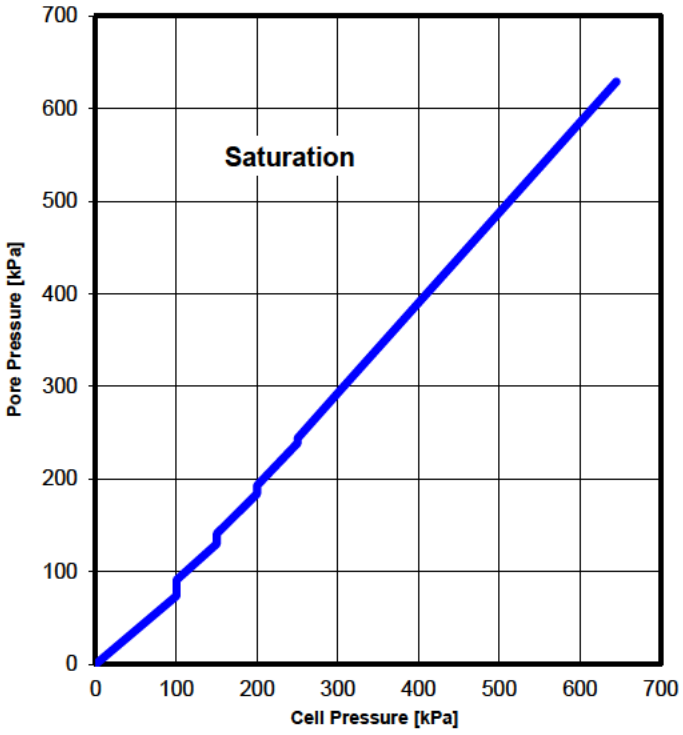
Cohesion c'	[kPa]	n/a
Angle of shearing resistance ϕ'	degrees	n/a

FINAL CONDITIONS (Calculated)

Moisture content	[%]	25.0		
Bulk density	[Mg/m^3]	1.99		

OTHER PARAMETERS

Secant modulus E' 50	[MPa]	76.6		
Strain at ε_{50}	[%]	0.63		



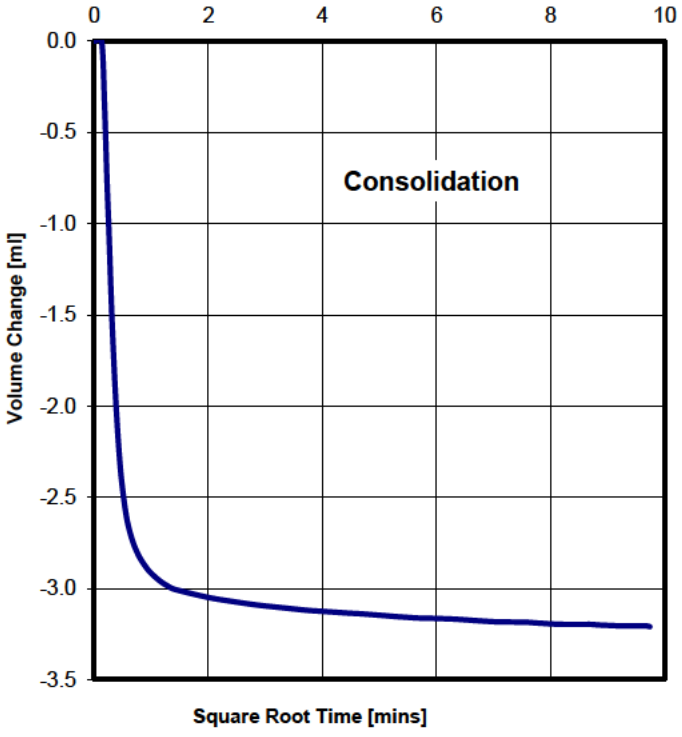


Figure Number	Sheet Number
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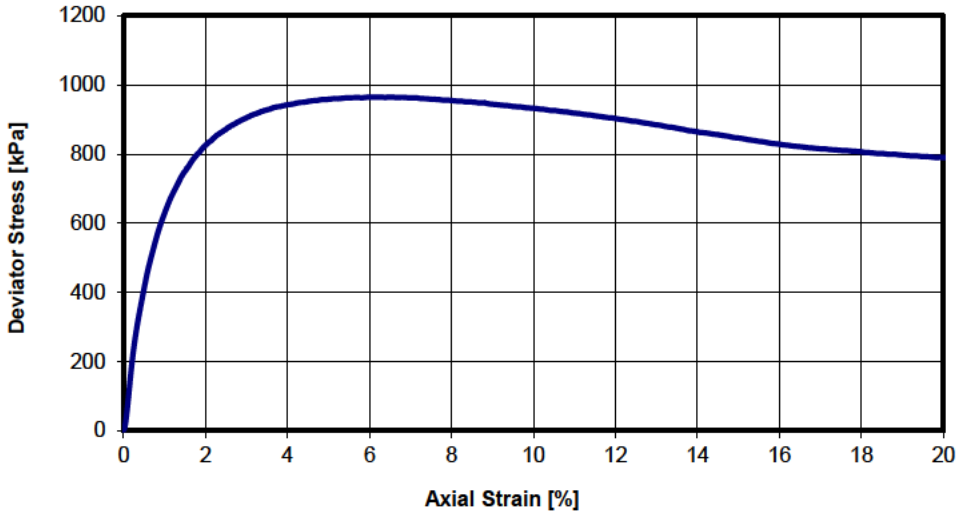
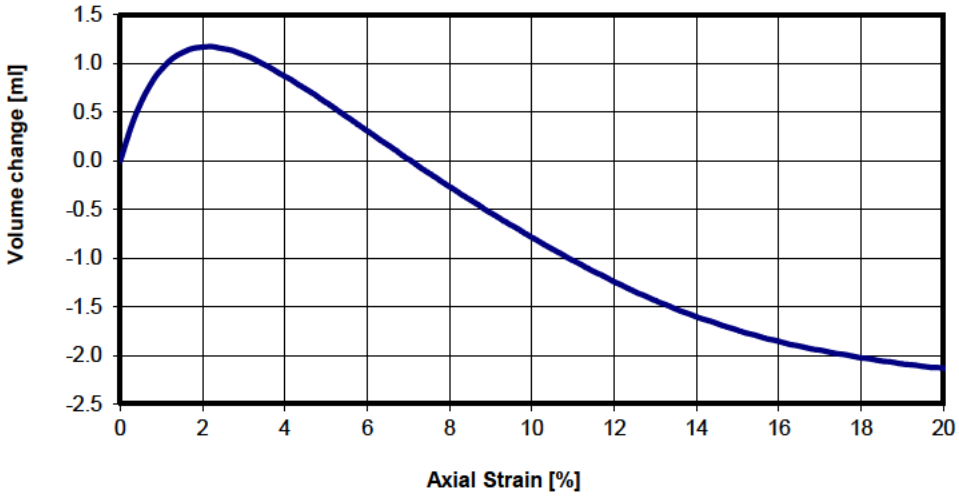
FUGRO	**ISOTROPICALLY CONSOLIDATED DRAINED TRIAXIAL COMPRESSION TEST : SINGLE SPECIMEN with measurement of volume change**	Project No.	F225236
		Borehole No.	NEPL7-BH01
		Sample No.	18BAGA
Project Name	Neptune L7 -17	Test type	CD





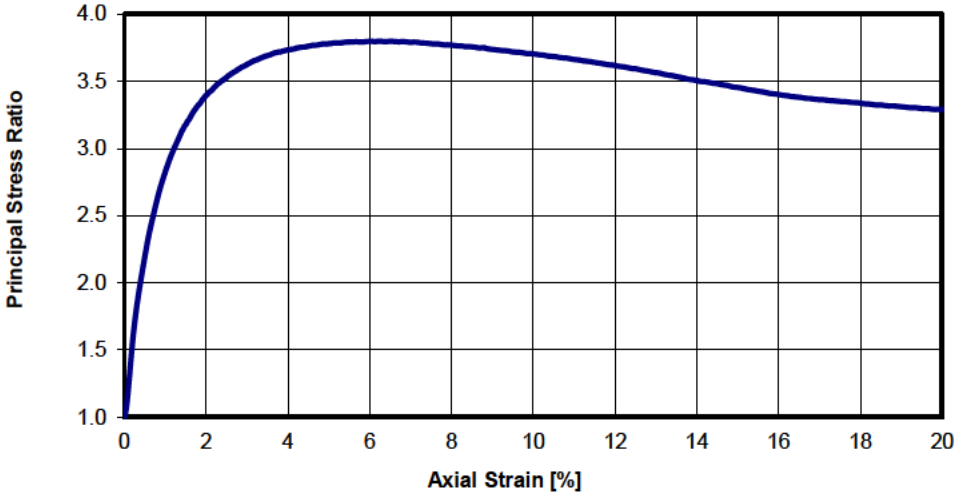
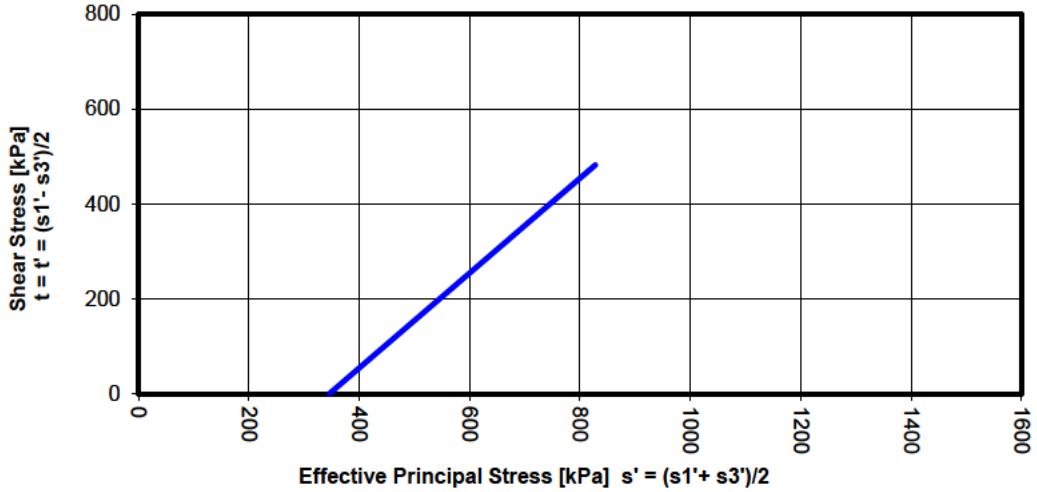
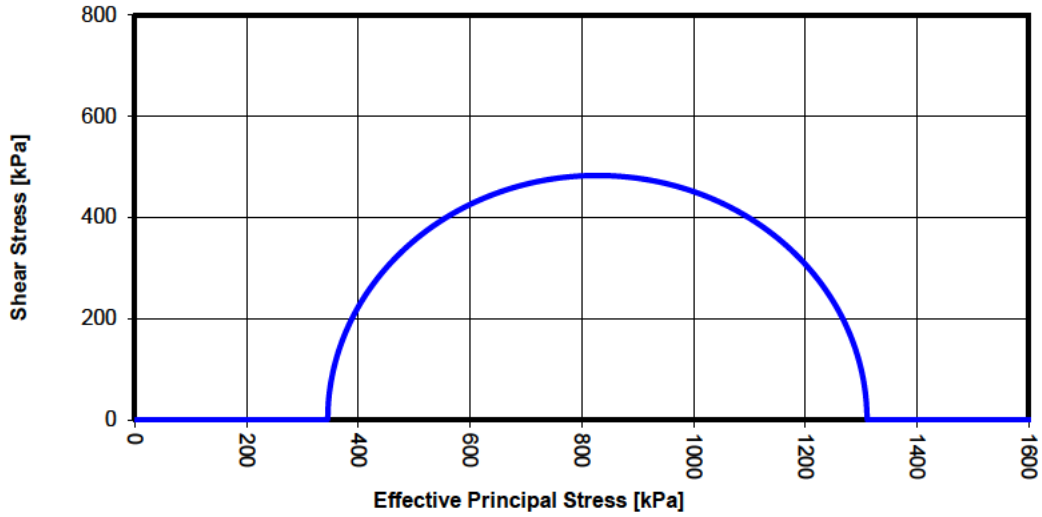


Figure Number	Sheet Number
	3/4

FUGRO	**ISOTROPICALLY CONSOLIDATED DRAINED TRIAXIAL COMPRESSION TEST : SINGLE SPECIMEN with measurement of volume change**	Project No.	F225236
		Borehole No.	NEPL7-BH01
		Sample No.	18BAGA
Project Name	Neptune L7 -17	Test type	CD





Mode of failure:

Compound

Figure Number	Sheet Number
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Contents Appendix D.5: Results of Ring Shear Tests

Page

D.5 Results of Ring Shear Tests 1
D.5.1 Comments on Results 1
D.5.2 References 1

List of Plates

Title	Plate
Results of Interface Ring Shear Tests	D.4-1 to D.4-20

D.5 Results of Ring Shear Tests

D.5.1 Comments on Results

Ring shear (RS) tests have been performed with Bromhead type test equipment, which is equipment most commonly used for such tests. Tests have been performed in accordance to ICP (Jardine et al., 2005) procedures. Bromhead ring shear equipment and mentioned method are in principle designed for testing of remoulded specimens of fine-grained cohesive soils and coarse-grained cohesionless soils up to a grain size of 425 µm. Tests performed on soils that fall outside the specified ranges of soil type and grain size may yield results outside expected ranges for the soils tested and/or may result in scatter of angle of residual shear resistance between multiple tests performed on material from similar samples.

Interface roughness of the soil-steel interface is measured before performing the test. No interface roughness is measured after performing the test

D.5.2 References

- Jardine, R., Chow, F., Overy, R. & Standing, J. (2005). *ICP design methods for driven piles in sands and clays*. Thomas Telford.

VISUAL DESCRIPTION
Dark grey fine to medium SAND

INITIAL		
Ext. Diameter of ring	[mm]	100.0
Int. Diameter of ring	[mm]	70.0
Weight of soil	[g]	35.07
Interface type		Soil - Steel
Interface roughness before	[µm]	8.06
Interface roughness after	[µm]	-
Water content	[%]	8.8
Bulk density	[Mg/m^3]	1.69
Dry density	[Mg/m^3]	1.55
% retained on 425 µm sieve	[%]	0.00

NOTES

TESTING PROCEDURES USED	
Consolidation	Jardine et al., 2005
Shearing	Jardine et al., 2005

QUALITY ASSURANCE	
Template Issue	1.0
Filename	NEPL7-BH01_06BAGA_RS01
Processed by	LB
Date	11/05/2023
Checked by	PH
Date	12/05/2023
Approved by	PH
Date	12/05/2023

TEST IDENTIFICATION		
Borehole		NEPL7-BH01
Sample		06BAGA
Depth	[m]	34.50
Test Number		RS01

DETERMINATION OF RESIDUAL STRENGTH BY SMALL RING SHEAR

CONSOLIDATION		
Consolidation pressure	[kPa]	200
Vertical displacement	[mm]	0.23
Vertical strain	[%]	4.66

FAST SHEAR STAGE		1
Consolidation pressure	[kPa]	200
Vertical displacement	[mm]	0.48
Vertical strain	[%]	9.66
Horizontal displacement	[mm]	49.99
Machine speed	[mm/min]	500
At peak stress		
Shear stress	[kPa]	119
Horizontal displacement	[mm]	25.00
Shear angle	[°]	30.8

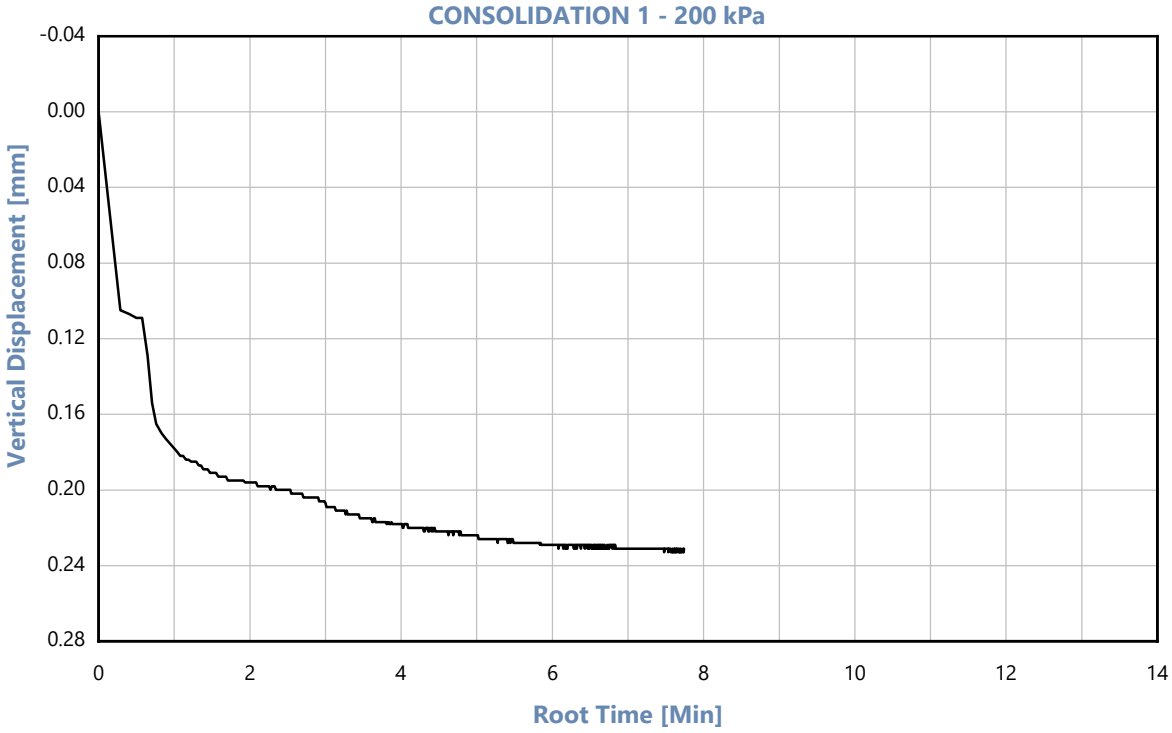
CONSOLIDATION		
Consolidation pressure	[kPa]	370
Vertical displacement	[mm]	0.65
Vertical strain	[%]	12.96

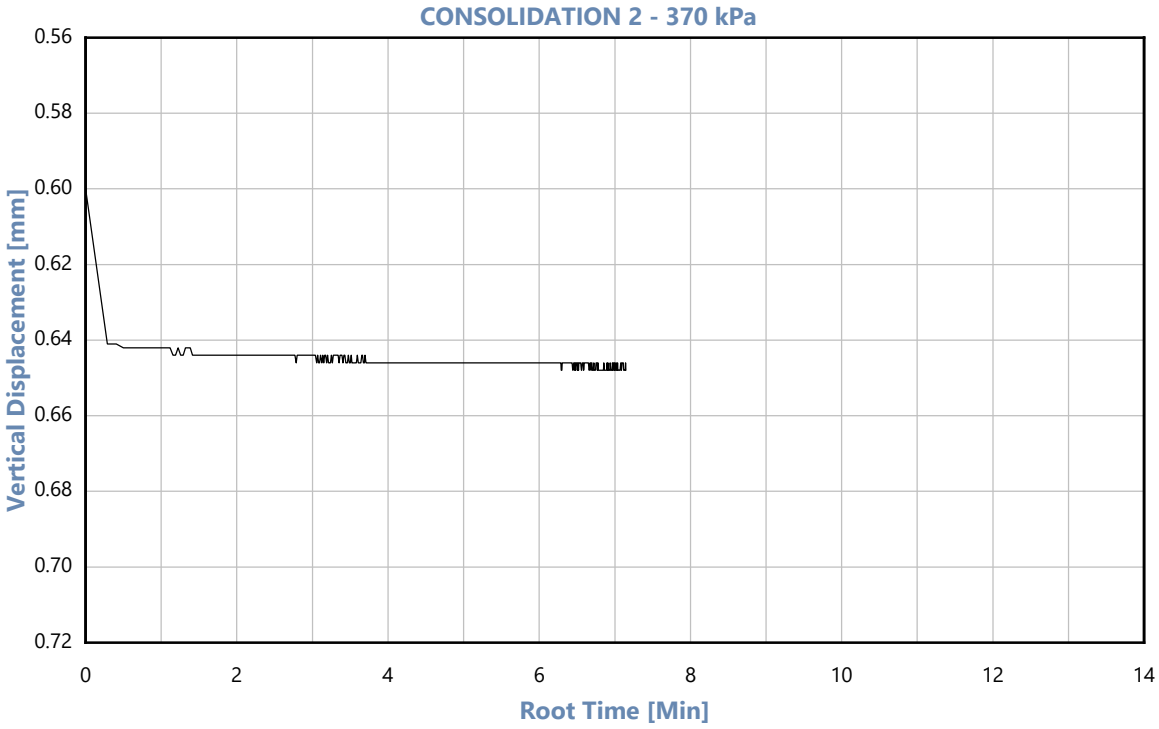
SLOW SHEAR STAGE		
Consolidation pressure	[kPa]	370
Vertical displacement	[mm]	0.77
Vertical strain	[%]	15.48
Horizontal displacement	[mm]	20.11
Machine speed	[mm/min]	0.018
At peak stress		
Shear stress	[kPa]	232
Horizontal displacement	[mm]	0.89
Shear angle	[°]	32.0
At residual stress		
Shear stress	[kPa]	222
Horizontal displacement	[mm]	20.11
Shear angle	[°]	31.0

TEST IDENTIFICATION		
Borehole		NEPL7-BH01
Sample		06BAGA
Depth	[m]	34.50
Test Number		RS01

DETERMINATION OF RESIDUAL STRENGTH BY SMALL RING SHEAR

Date: 11/05/2023
5.
Drawn By
Filename: 230238-F225236 \ SHEAR \ NEPL7-BH01_06BAGA_RS01.OPJ
Template Issue: 2
Date: 12/05/2023
Date: 12/05/2023
5.
Checked By
5.
Approved By





TEST IDENTIFICATION		
Borehole	NEPL7-BH01	Interface Roughness Before (µm) : 8.06
Sample	06BAGA	
Depth [m]	34.50	Interface Roughness After (µm) : --
Test number	RS01	

RING SHEAR TEST - CONSOLIDATION
SOIL / STEEL INTERFACE

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Template Issue: 2
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Checked By
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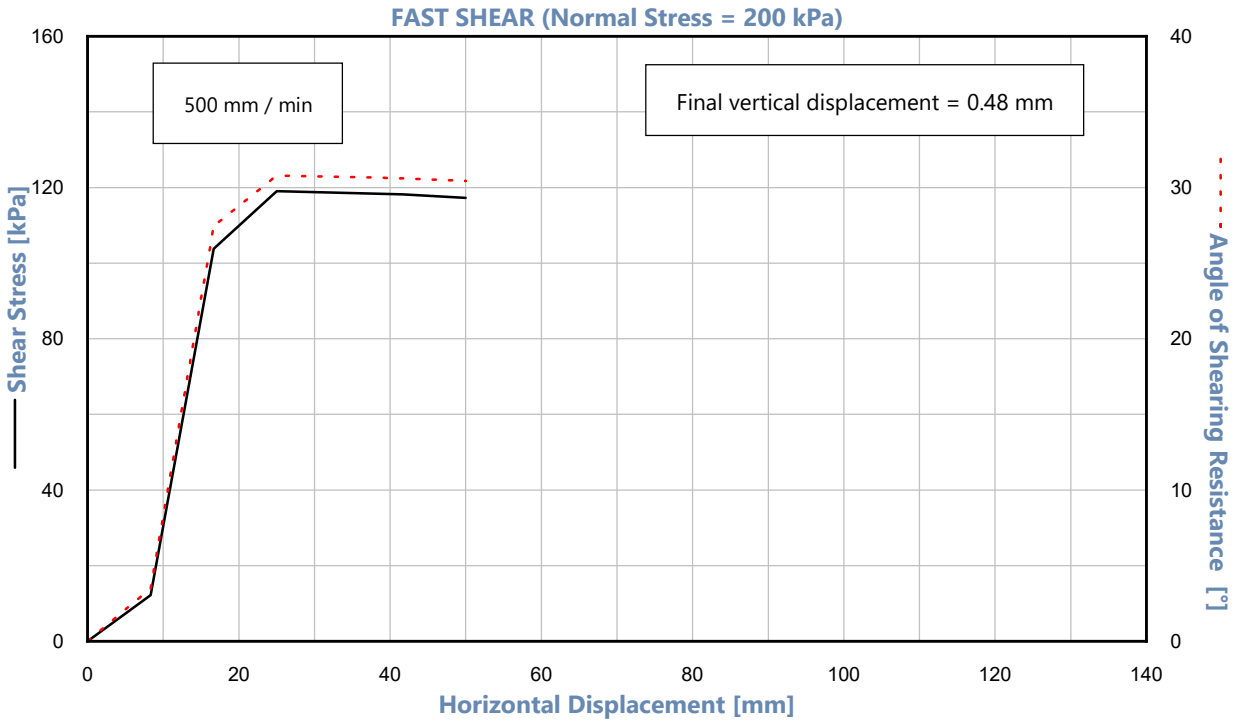
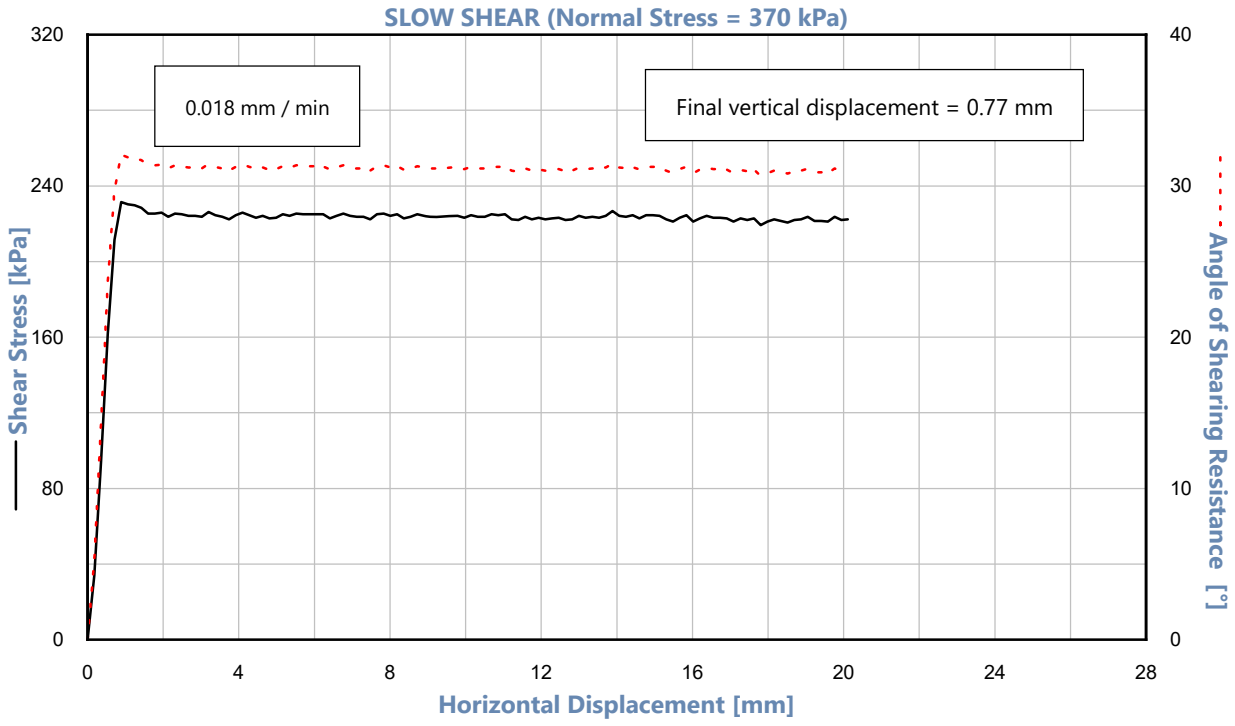
Approved By :





TEST IDENTIFICATION		
Borehole	NEPL7-BH01	Interface Roughness Before (µm) : 8.06
Sample	06BAGA	
Depth [m]	34.50	Interface Roughness After (µm) : --
Test number	RS01	

RING SHEAR TEST - SHEARING
SOIL / STEEL INTERFACE

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Drawn By
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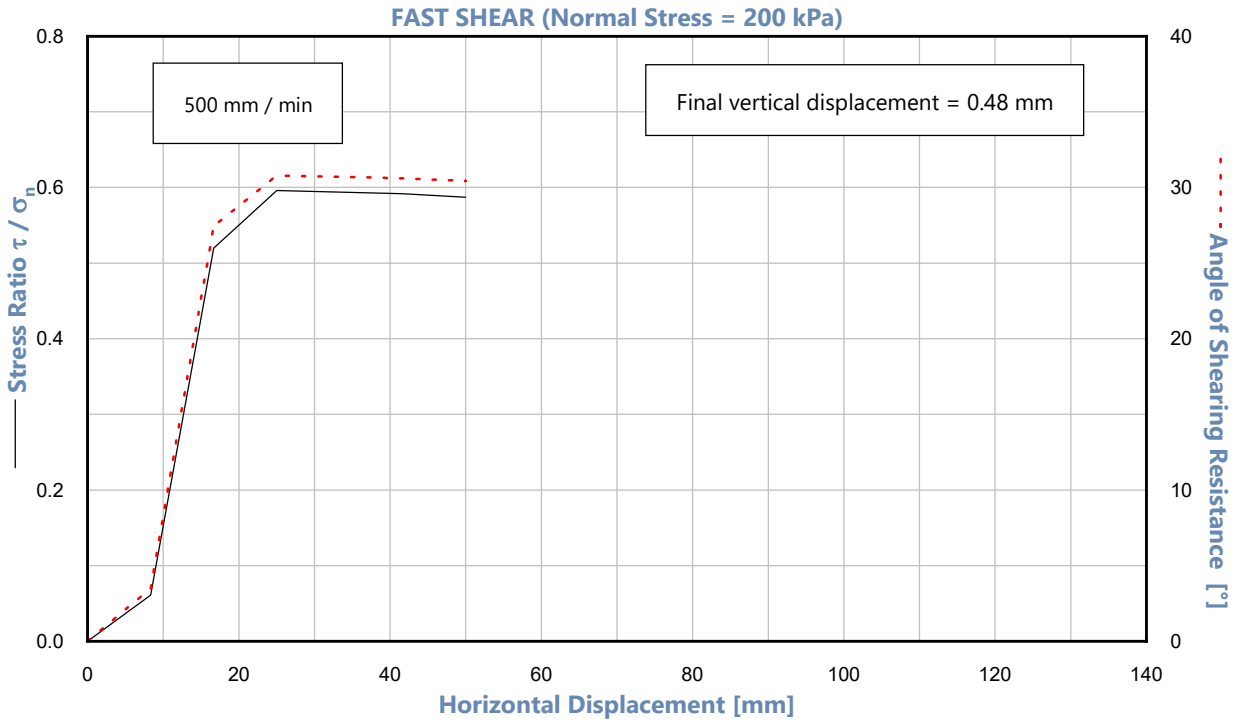

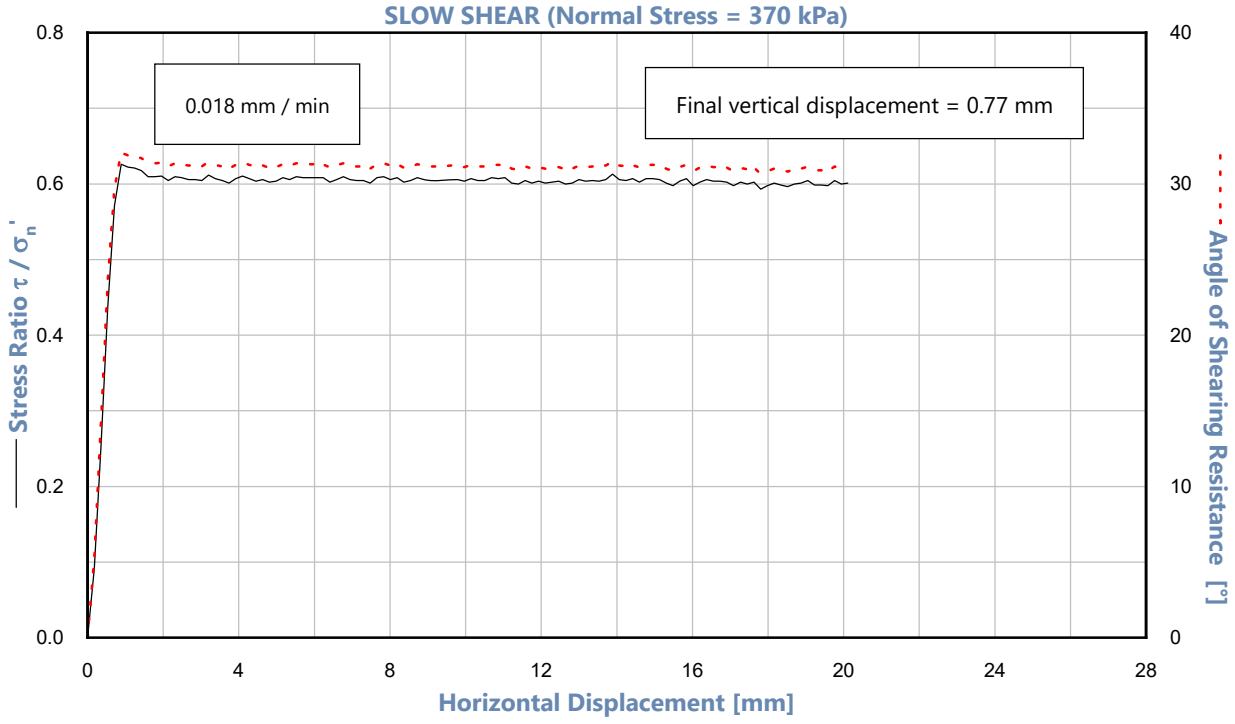

TEST IDENTIFICATION		
Borehole	NEPL7-BH01	Interface Roughness Before (µm) : 8.06
Sample	06BAGA	
Depth [m]	34.50	Interface Roughness After (µm) : --
Test number	RS01	

RING SHEAR TEST - SHEARING
SOIL / STEEL INTERFACE

VISUAL DESCRIPTION
Dark grey fine SAND

INITIAL		
Ext. Diameter of ring	[mm]	100.0
Int. Diameter of ring	[mm]	70.0
Weight of soil	[g]	34.35
Interface type		Soil - Steel
Interface roughness before	[µm]	8.26
Interface roughness after	[µm]	-
Water content	[%]	8.7
Bulk density	[Mg/m^3]	1.69
Dry density	[Mg/m^3]	1.55
% retained on 425 µm sieve	[%]	0.00

NOTES

TESTING PROCEDURES USED	
Consolidation	Jardine et al., 2005
Shearing	Jardine et al., 2005

QUALITY ASSURANCE	
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Filename	NEPL7-BH01_08BAGB_RS02
Processed by	5.1
Date	11/05/2023
Checked by	5.1
Date	12/05/2023
Approved by	5.1
Date	12/05/2023

TEST IDENTIFICATION		
Borehole		NEPL7-BH01
Sample		08BAGB
Depth	[m]	38.15
Test Number		RS02

DETERMINATION OF RESIDUAL STRENGTH BY SMALL RING SHEAR

CONSOLIDATION		
Consolidation pressure	[kPa]	210
Vertical displacement	[mm]	0.06
Vertical strain	[%]	1.40

FAST SHEAR STAGE		1
Consolidation pressure	[kPa]	210
Vertical displacement	[mm]	0.50
Vertical strain	[%]	10.00
Horizontal displacement	[mm]	49.99
Machine speed	[mm/min]	500
At peak stress		
Shear stress	[kPa]	121
Horizontal displacement	[mm]	41.66
Shear angle	[°]	29.9

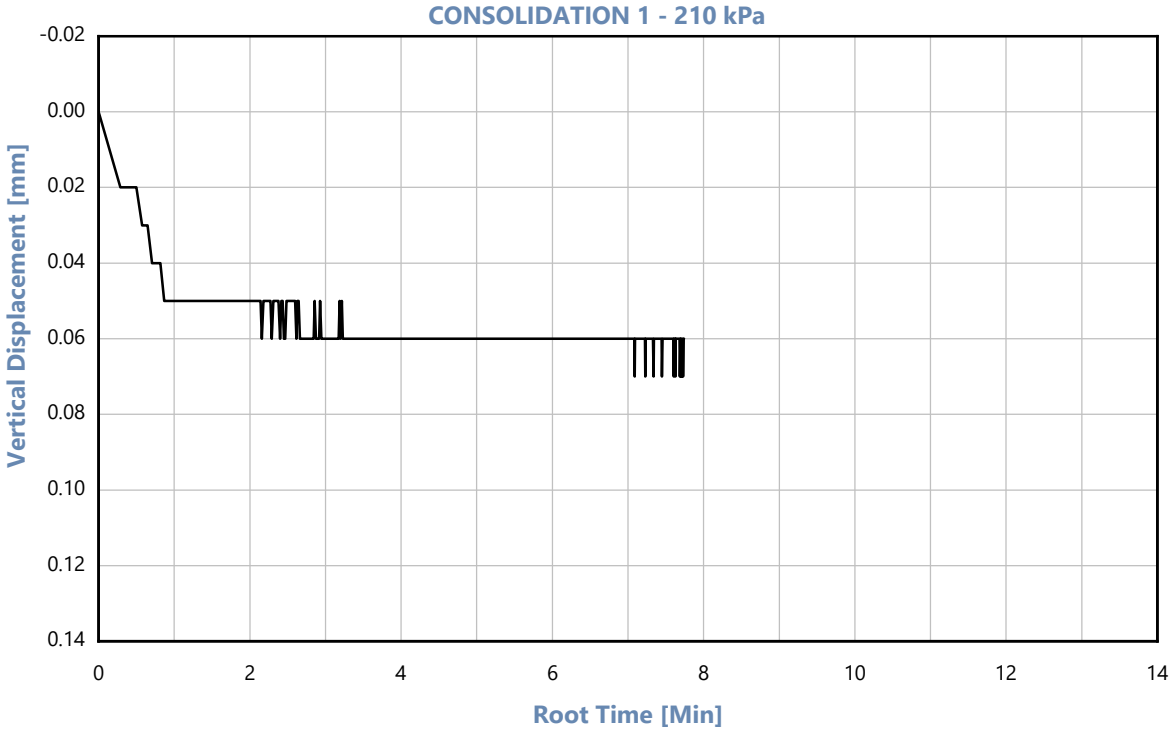
CONSOLIDATION		
Consolidation pressure	[kPa]	340
Vertical displacement	[mm]	0.75
Vertical strain	[%]	15.00

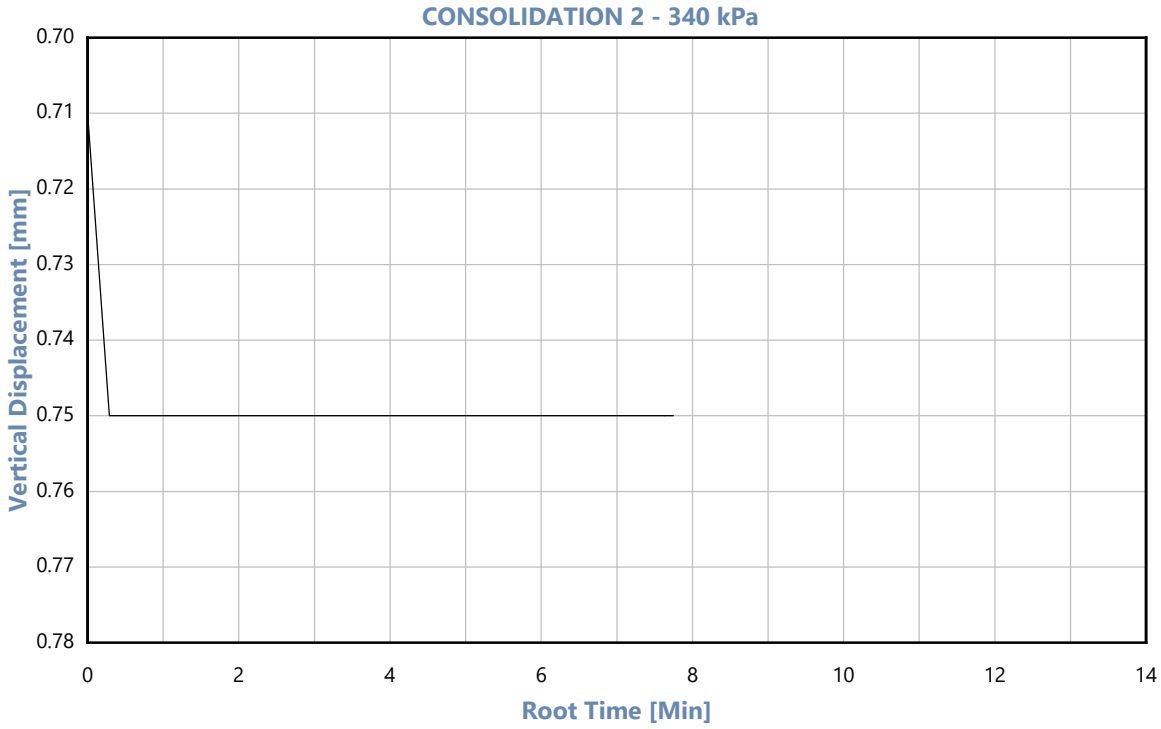
SLOW SHEAR STAGE		
Consolidation pressure	[kPa]	340
Vertical displacement	[mm]	0.97
Vertical strain	[%]	19.40
Horizontal displacement	[mm]	20.13
Machine speed	[mm/min]	0.018
At peak stress		
Shear stress	[kPa]	221
Horizontal displacement	[mm]	11.39
Shear angle	[°]	33.0
At residual stress		
Shear stress	[kPa]	213
Horizontal displacement	[mm]	20.13
Shear angle	[°]	32.1

TEST IDENTIFICATION		
Borehole		NEPL7-BH01
Sample		08BAGB
Depth	[m]	38.15
Test Number		RS02

DETERMINATION OF RESIDUAL STRENGTH BY SMALL RING SHEAR

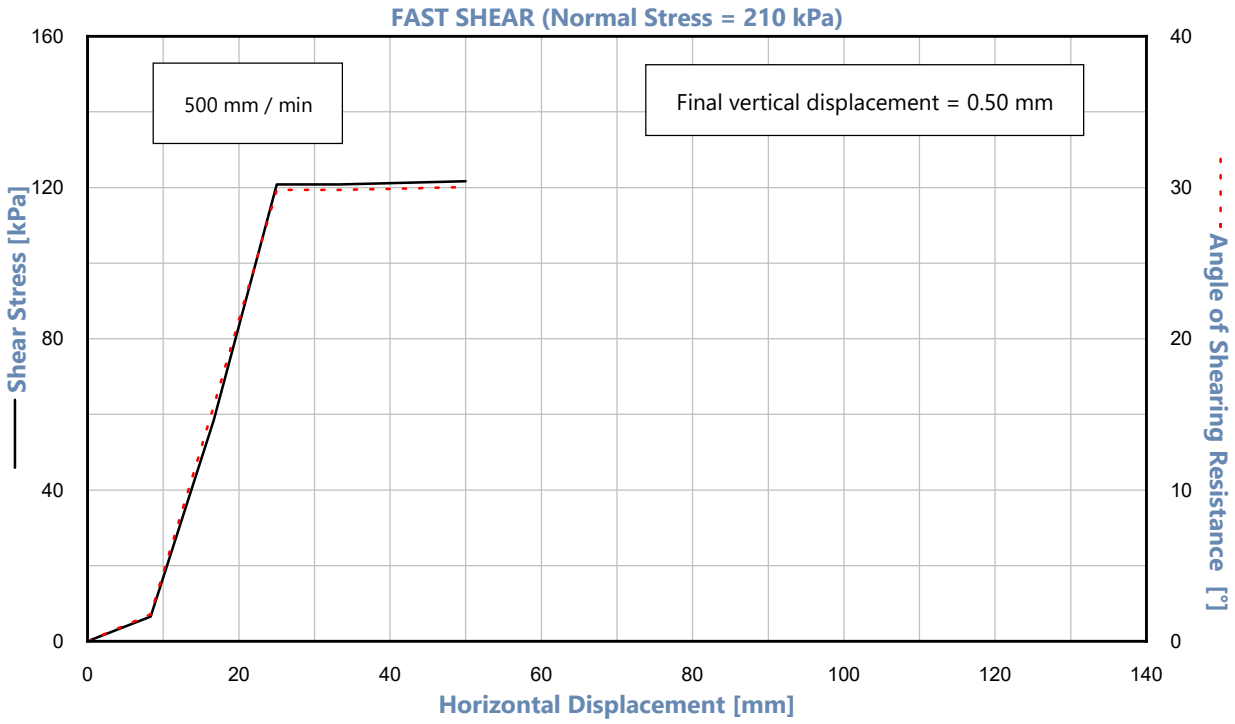
Date: 11/05/2023
Drawn By: 5.
Filename: 230238-F225236 \ SHEAR \ NEPL7-BH01_08BAGB_RS02.OPJ
Template Issue: 2
Date: 12/05/2023
Date: 12/05/2023
Checked By: 5.
Approved By: 5.

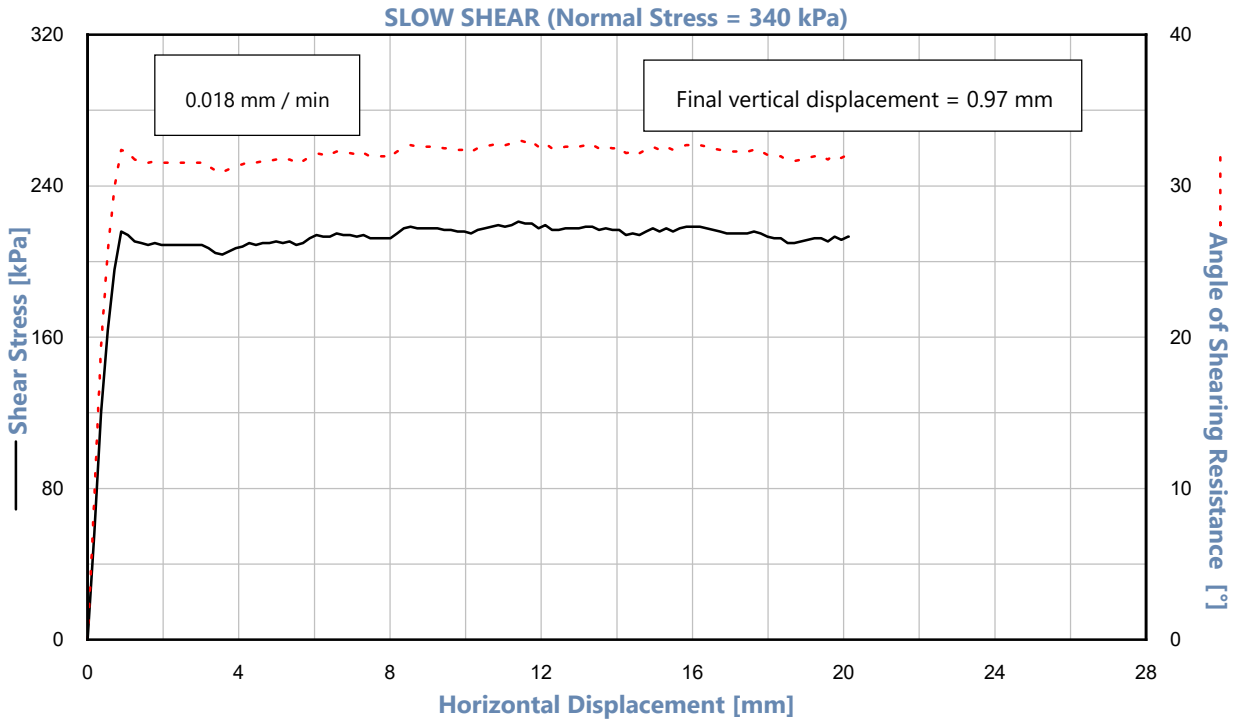




TEST IDENTIFICATION		
Borehole	NEPL7-BH01	Interface Roughness Before (µm) : 8.26
Sample	08BAGB	
Depth [m]	38.15	Interface Roughness After (µm) : --
Test number	RS02	

RING SHEAR TEST - CONSOLIDATION
SOIL / STEEL INTERFACE





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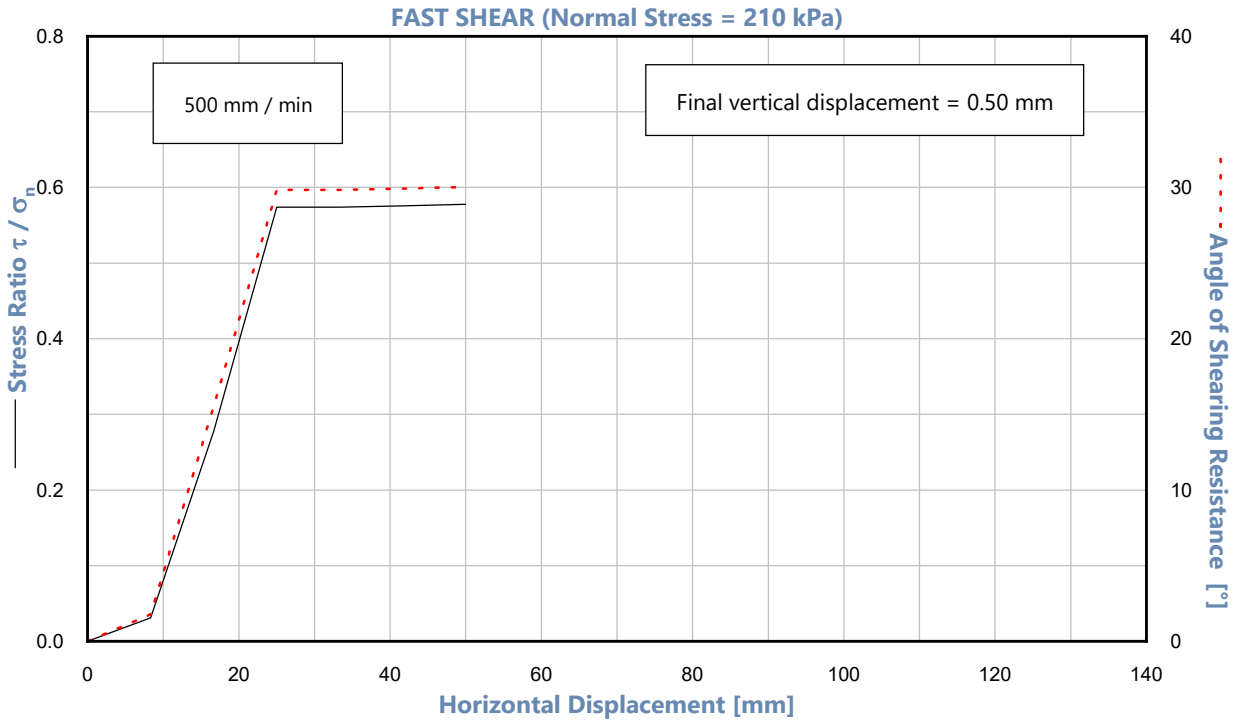
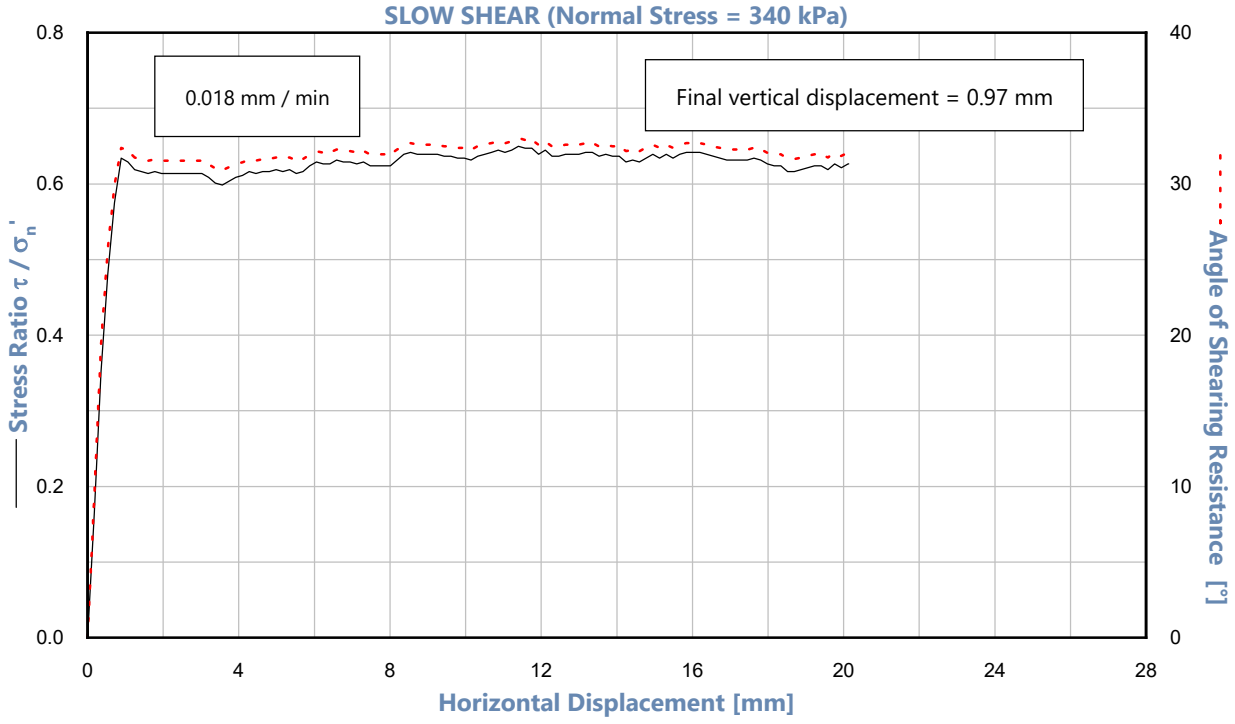
Template Issue: 2
Filename: 230238-F225236 \ SHEAR \ NEPL7-BH01_08BAGB_RS02.OPJ
Date: 12/05/2023
Date: 12/05/2023
Checked By 5.
Approved By 5.

TEST IDENTIFICATION		
Borehole	NEPL7-BH01	Interface Roughness Before (µm) : 8.26
Sample	08BAGB	
Depth [m]	38.15	Interface Roughness After (µm) : --
Test number	RS02	

RING SHEAR TEST - SHEARING
SOIL / STEEL INTERFACE

Date: 11/05/2023
5.
Drawn By
Template Issue: 2
Filename: 230238-F225236 \ SHEAR \ NEPL7-BH01_08BAGB_RS02.OPJ
Date: 12/05/2023
Date: 12/05/2023
5.
Checked By :
5.
Approved By





TEST IDENTIFICATION		
Borehole	NEPL7-BH01	Interface Roughness Before (µm) : 8.26
Sample	08BAGB	
Depth [m]	38.15	Interface Roughness After (µm) : --
Test number	RS02	

RING SHEAR TEST - SHEARING
SOIL / STEEL INTERFACE

VISUAL DESCRIPTION
Olive fine SAND

INITIAL		
Ext. Diameter of ring	[mm]	100.0
Int. Diameter of ring	[mm]	70.0
Weight of soil	[g]	38.92
Interface type		Soil - Steel
Interface roughness before	[µm]	8.22
Interface roughness after	[µm]	-
Water content	[%]	9.9
Bulk density	[Mg/m^3]	1.86
Dry density	[Mg/m^3]	1.70
% retained on 425 µm sieve	[%]	0.00

NOTES

TESTING PROCEDURES USED	
Consolidation	Jardine et al., 2005
Shearing	Jardine et al., 2005

QUALITY ASSURANCE	
Template Issue	1.0
Filename	NEPL7-BH01_11BAGA_RS03
Processed by	5.1
Date	11/05/2023
Checked by	5.1
Date	12/05/2023
Approved by	5.1
Date	12/05/2023

TEST IDENTIFICATION		
Borehole		NEPL7-BH01
Sample		11BAGA
Depth	[m]	43.00
Test Number		RS03

DETERMINATION OF RESIDUAL STRENGTH BY SMALL RING SHEAR

CONSOLIDATION		
Consolidation pressure	[kPa]	185
Vertical displacement	[mm]	0.15
Vertical strain	[%]	3.02

FAST SHEAR STAGE		1
Consolidation pressure	[kPa]	185
Vertical displacement	[mm]	0.47
Vertical strain	[%]	9.34
Horizontal displacement	[mm]	41.66
Machine speed	[mm/min]	500
At peak stress		
Shear stress	[kPa]	110
Horizontal displacement	[mm]	25.00
Shear angle	[°]	30.7

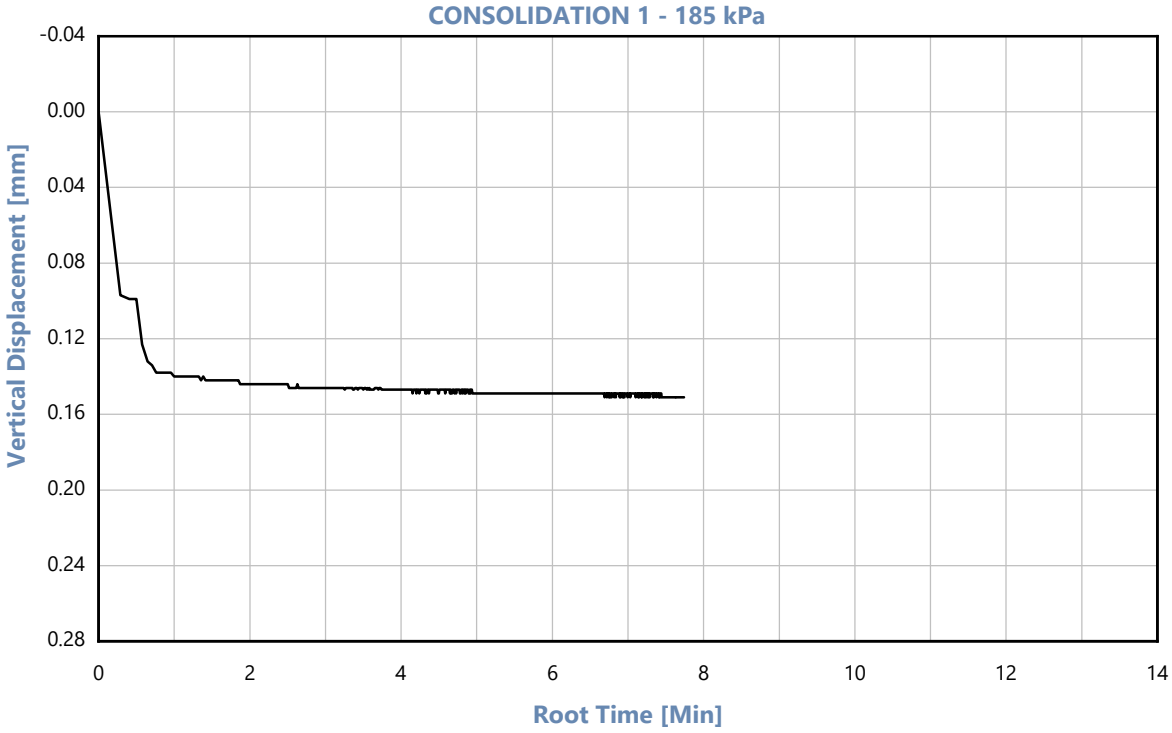
CONSOLIDATION		
Consolidation pressure	[kPa]	830
Vertical displacement	[mm]	0.79
Vertical strain	[%]	15.80

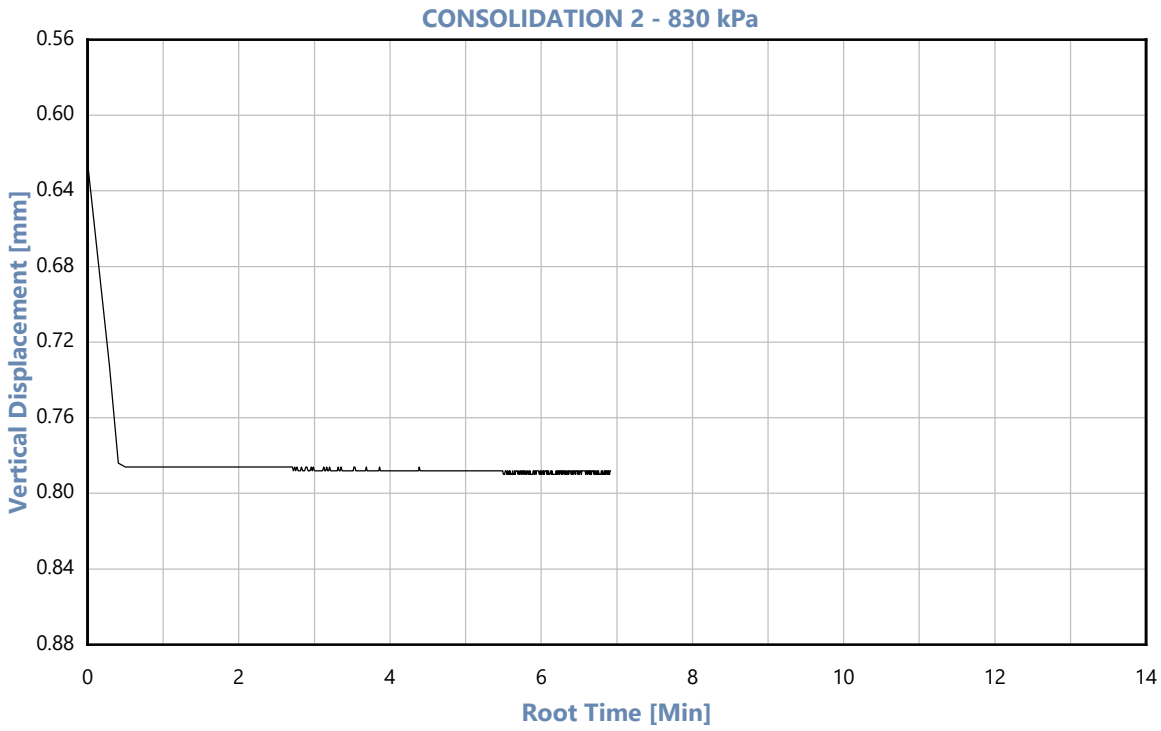
SLOW SHEAR STAGE		
Consolidation pressure	[kPa]	830
Vertical displacement	[mm]	0.99
Vertical strain	[%]	19.78
Horizontal displacement	[mm]	20.12
Machine speed	[mm/min]	0.018
At peak stress		
Shear stress	[kPa]	535
Horizontal displacement	[mm]	2.14
Shear angle	[°]	32.8
At residual stress		
Shear stress	[kPa]	524
Horizontal displacement	[mm]	20.12
Shear angle	[°]	32.3

TEST IDENTIFICATION		
Borehole		NEPL7-BH01
Sample		11BAGA
Depth	[m]	43.00
Test Number		RS03

DETERMINATION OF RESIDUAL STRENGTH BY SMALL RING SHEAR

Date: 11/05/2023
Drawn By 5.
Filename: 230238-F225236 \ SHEAR \ NEPL7-BH01_11BAGA_RS03.OPJ
Template Issue: 2
Date: 12/05/2023
Checked By 5.
Date: 12/05/2023
Approved By 5.





TEST IDENTIFICATION		
Borehole	NEPL7-BH01	Interface Roughness Before (µm) : 8.22
Sample	11BAGA	
Depth [m]	43.00	Interface Roughness After (µm) : --
Test number	RS03	

RING SHEAR TEST - CONSOLIDATION
SOIL / STEEL INTERFACE

Date: 11/05/2023
Drawn By: 5.
Template Issue: 2
Filename: 230238-F225236 \ SHEAR \ NEPL7-BH01_11BAGA_RS03.OPJ
Date: 12/05/2023
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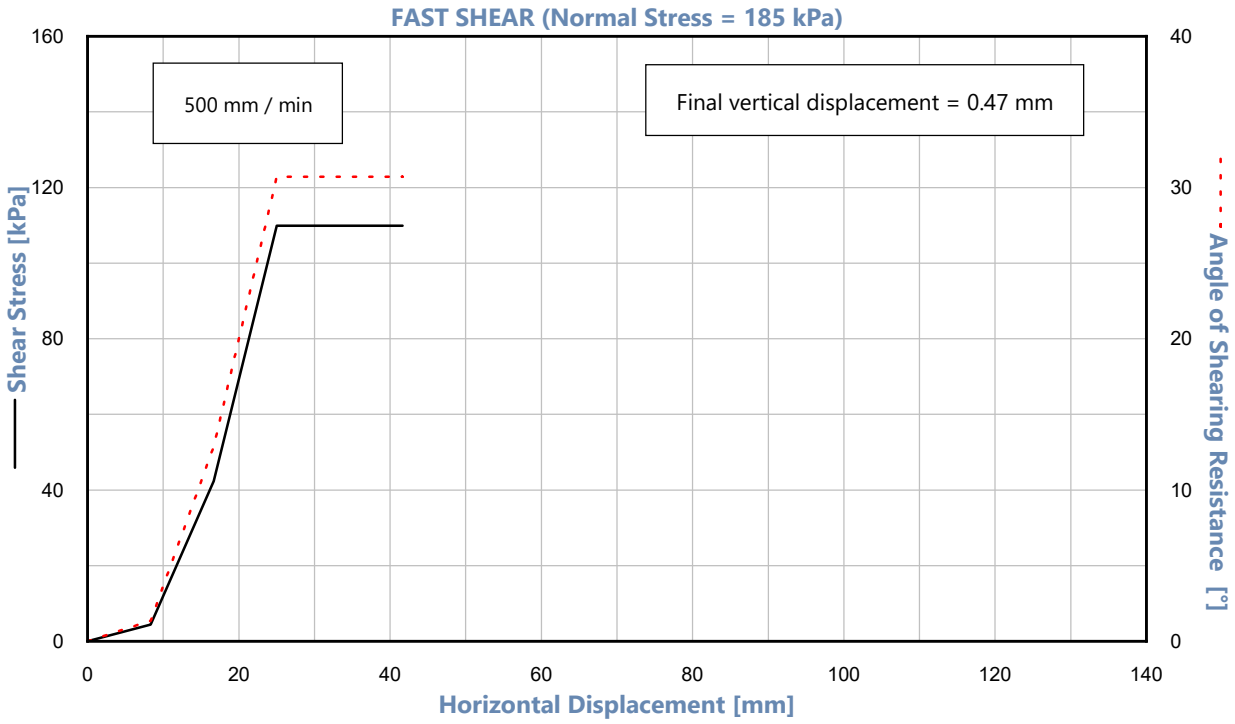

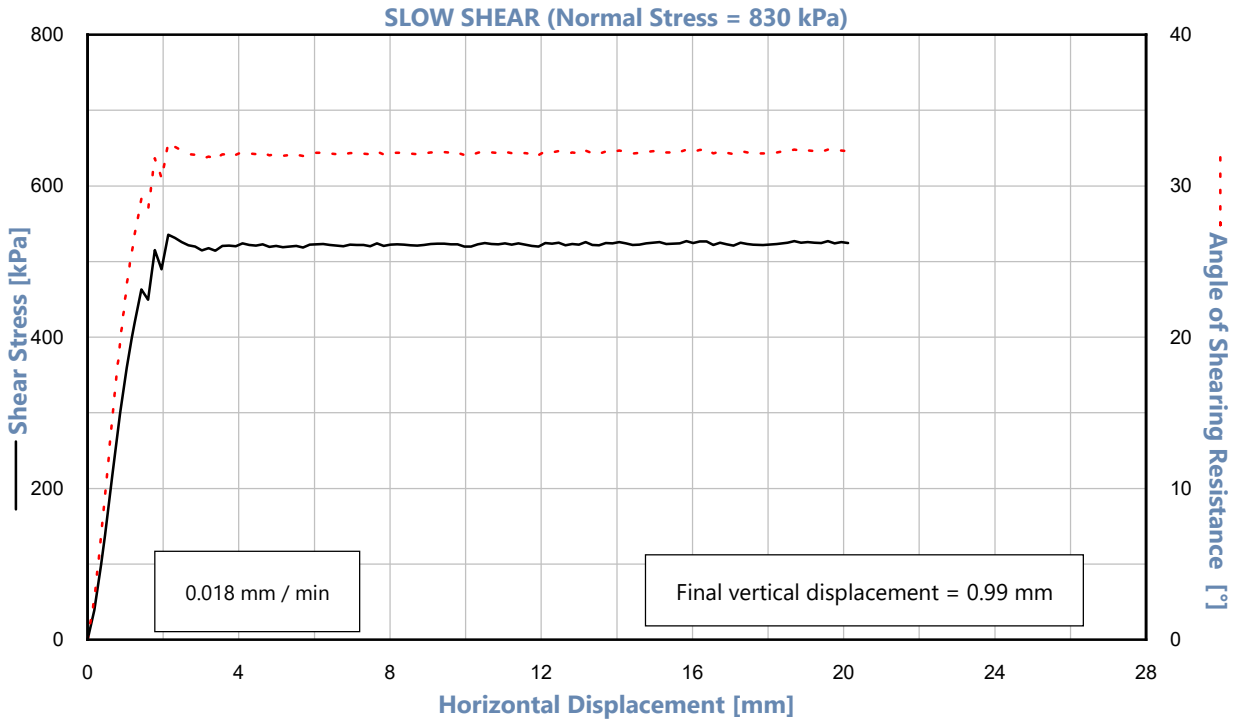

TEST IDENTIFICATION		
Borehole	NEPL7-BH01	Interface Roughness Before (µm) : 8.22
Sample	11BAGA	
Depth [m]	43.00	Interface Roughness After (µm) : --
Test number	RS03	

RING SHEAR TEST - SHEARING
SOIL / STEEL INTERFACE

Date: 11/05/2023
Drawn By: 5.
Template Issue: 2
Filename: 230238-F225236 \ SHEAR \ NEPL7-BH01_11BAGA_RS03.OPJ
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Checked By: 5.
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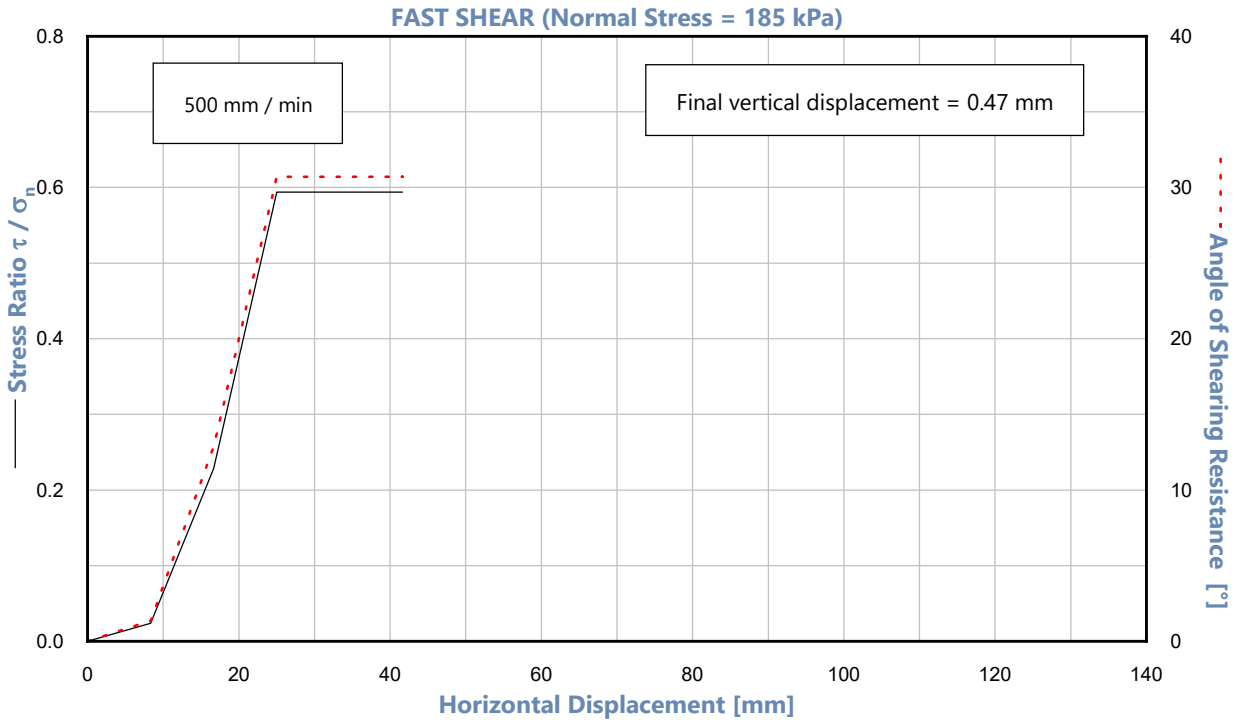

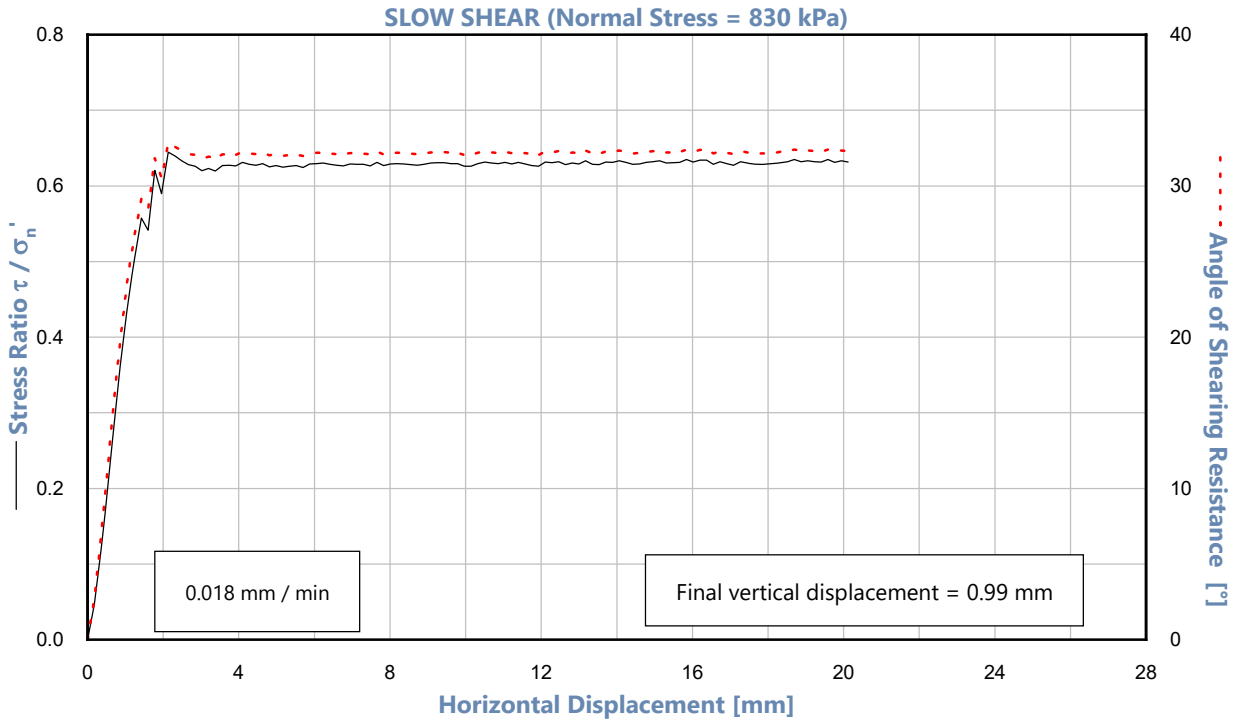

TEST IDENTIFICATION		
Borehole	NEPL7-BH01	Interface Roughness Before (µm) : 8.22
Sample	11BAGA	
Depth [m]	43.00	Interface Roughness After (µm) : --
Test number	RS03	

RING SHEAR TEST - SHEARING
SOIL / STEEL INTERFACE

VISUAL DESCRIPTION
Dark grey fine SAND

INITIAL		
Ext. Diameter of ring	[mm]	100.0
Int. Diameter of ring	[mm]	70.0
Weight of soil	[g]	36.85
Interface type		Soil - Steel
Interface roughness before	[µm]	8.25
Interface roughness after	[µm]	-
Water content	[%]	8.2
Bulk density	[Mg/m^3]	1.81
Dry density	[Mg/m^3]	1.67
% retained on 425 µm sieve	[%]	0.00

NOTES

TESTING PROCEDURES USED	
Consolidation	Jardine et al., 2005
Shearing	Jardine et al., 2005

QUALITY ASSURANCE	
Template Issue	1.0
Filename	NEPL7-BH01_17BAGA_RS04
Processed by	5.
Date	11/05/2023
Checked by	5.
Date	12/05/2023
Approved by	5.
Date	12/05/2023

TEST IDENTIFICATION		
Borehole		NEPL7-BH01
Sample		17BAGA
Depth	[m]	58.00
Test Number		RS04

DETERMINATION OF RESIDUAL STRENGTH BY SMALL RING SHEAR

CONSOLIDATION		
Consolidation pressure	[kPa]	240
Vertical displacement	[mm]	0.09
Vertical strain	[%]	2.02

FAST SHEAR STAGE		1
Consolidation pressure	[kPa]	240
Vertical displacement	[mm]	0.42
Vertical strain	[%]	8.34
Horizontal displacement	[mm]	58.33
Machine speed	[mm/min]	500
At peak stress		
Shear stress	[kPa]	160
Horizontal displacement	[mm]	49.99
Shear angle	[°]	33.7

CONSOLIDATION		
Consolidation pressure	[kPa]	670
Vertical displacement	[mm]	0.63
Vertical strain	[%]	12.62

SLOW SHEAR STAGE		
Consolidation pressure	[kPa]	670
Vertical displacement	[mm]	0.87
Vertical strain	[%]	17.44
Horizontal displacement	[mm]	20.45
Machine speed	[mm/min]	0.018
At peak stress		
Shear stress	[kPa]	480
Horizontal displacement	[mm]	2.31
Shear angle	[°]	35.6
At residual stress		
Shear stress	[kPa]	456
Horizontal displacement	[mm]	20.45
Shear angle	[°]	34.2

TEST IDENTIFICATION		
Borehole		NEPL7-BH01
Sample		17BAGA
Depth	[m]	58.00
Test Number		RS04

DETERMINATION OF RESIDUAL STRENGTH BY SMALL RING SHEAR

Date: 11/05/2023

5.

Drawn By :

Filename: 230238-F225236 \ SHEAR \ NEPL7-BH01_17BAGA_RS04.OPJ

Template Issue: 2

Date: 12/05/2023

Date: 12/05/2023

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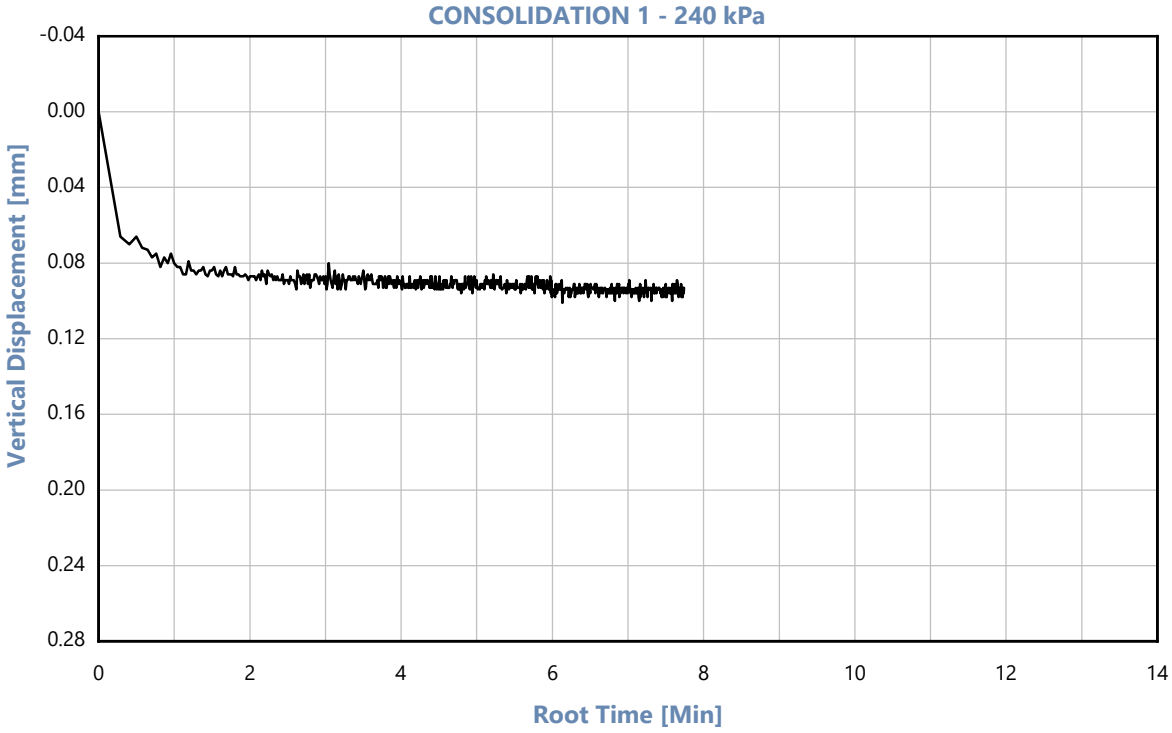
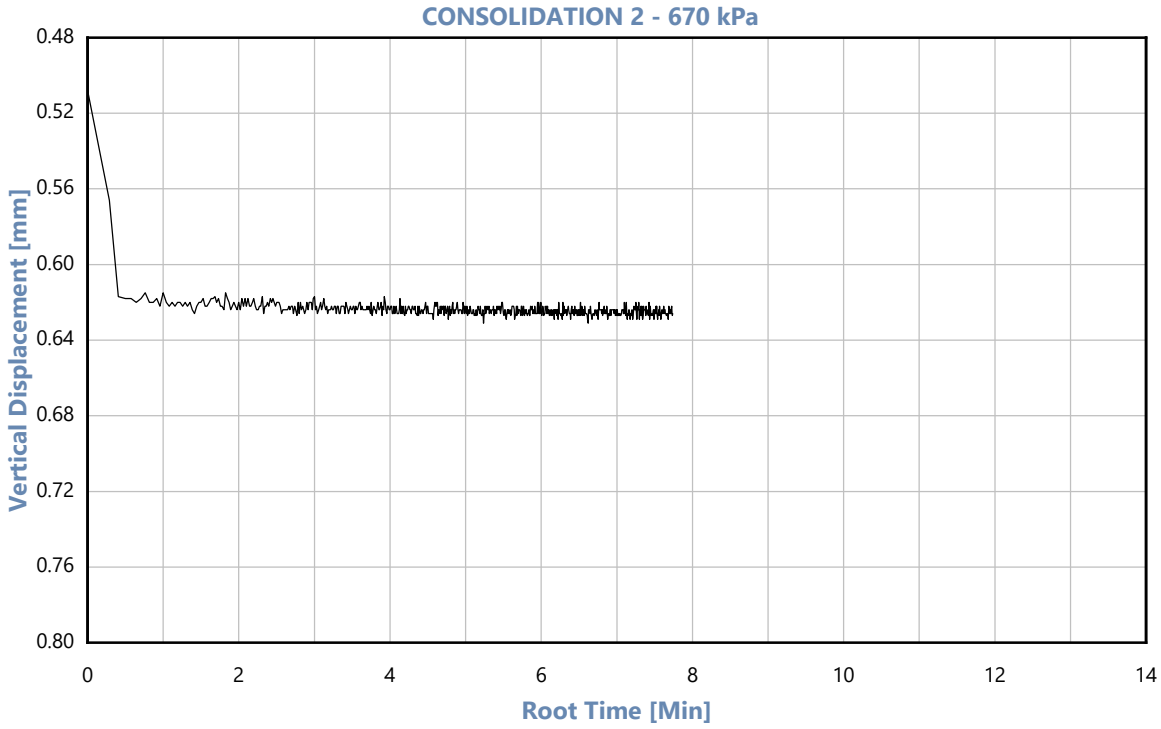
Checked By :

Approved By :





TEST IDENTIFICATION		
Borehole	NEPL7-BH01	Interface Roughness Before (µm) : 8.25
Sample	17BAGA	
Depth [m]	58.00	Interface Roughness After (µm) : --
Test number	RS04	

RING SHEAR TEST - CONSOLIDATION
SOIL / STEEL INTERFACE

Date: 11/05/2023
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Filename: 230238-F225236 \ SHEAR \ NEPL7-BH01_17BAGA_RS04.OPJ
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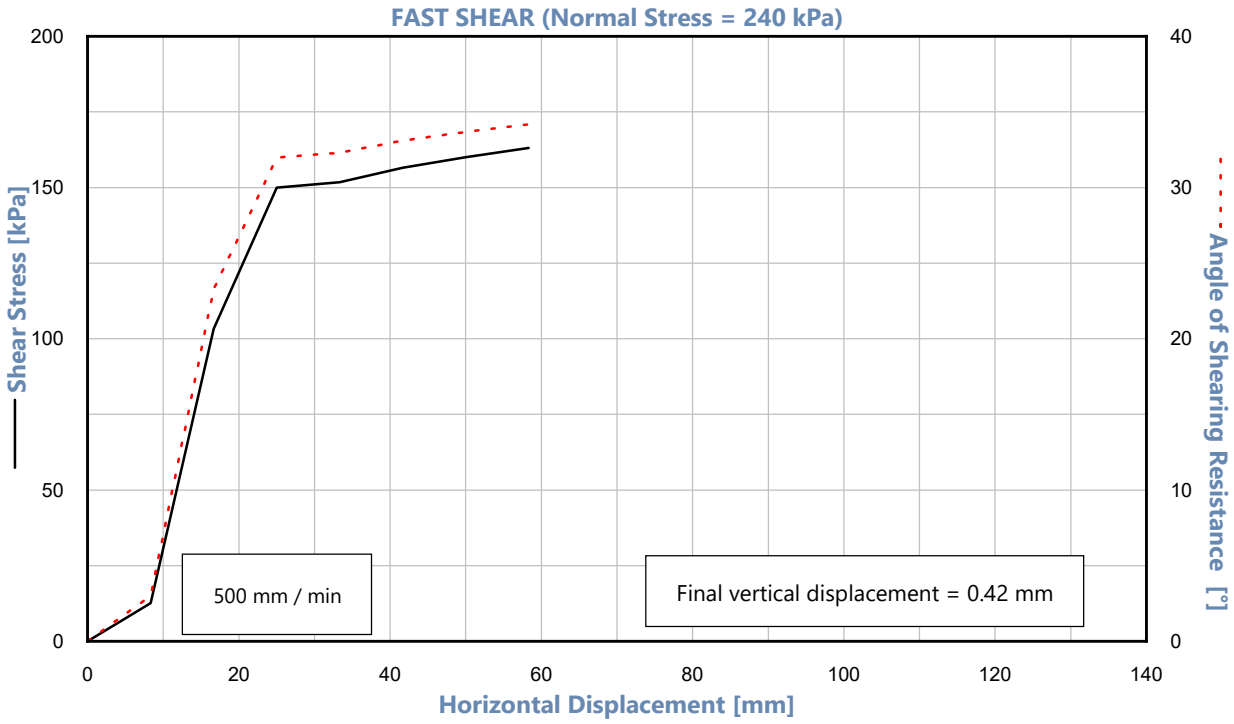

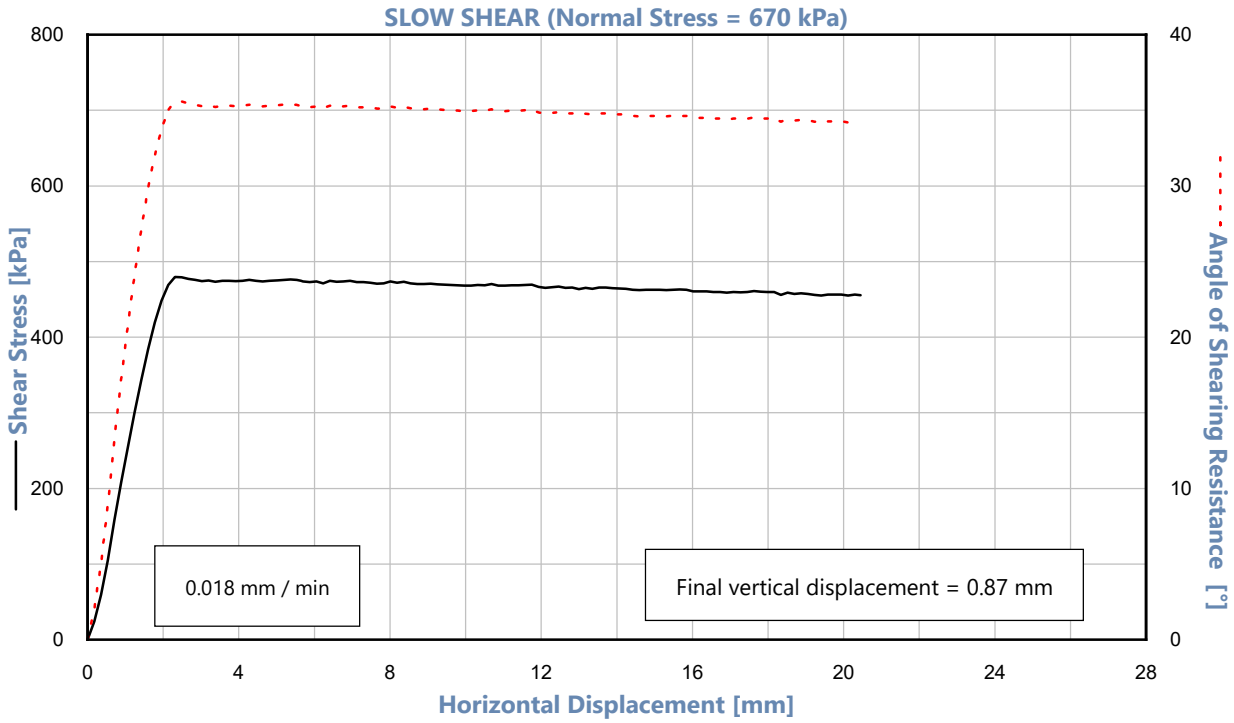

TEST IDENTIFICATION		
Borehole	NEPL7-BH01	Interface Roughness Before (µm) : 8.25
Sample	17BAGA	
Depth [m]	58.00	Interface Roughness After (µm) : --
Test number	RS04	

RING SHEAR TEST - SHEARING
SOIL / STEEL INTERFACE

Date: 11/05/2023
5.
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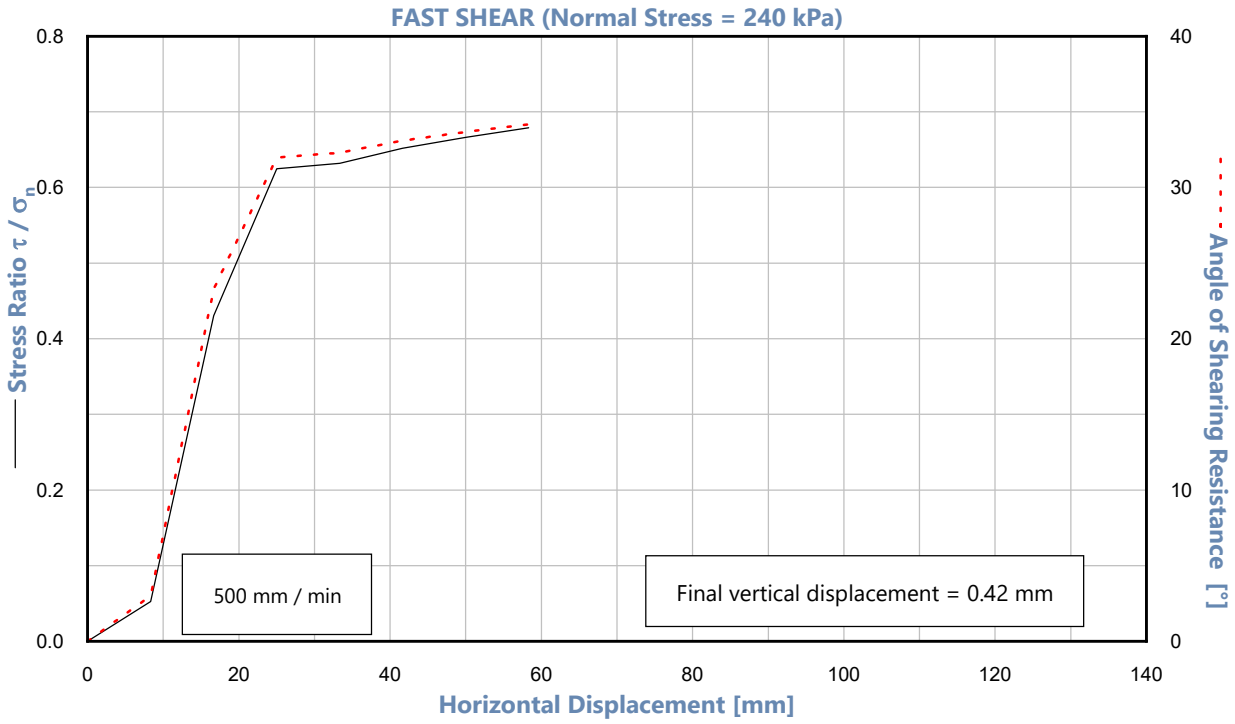
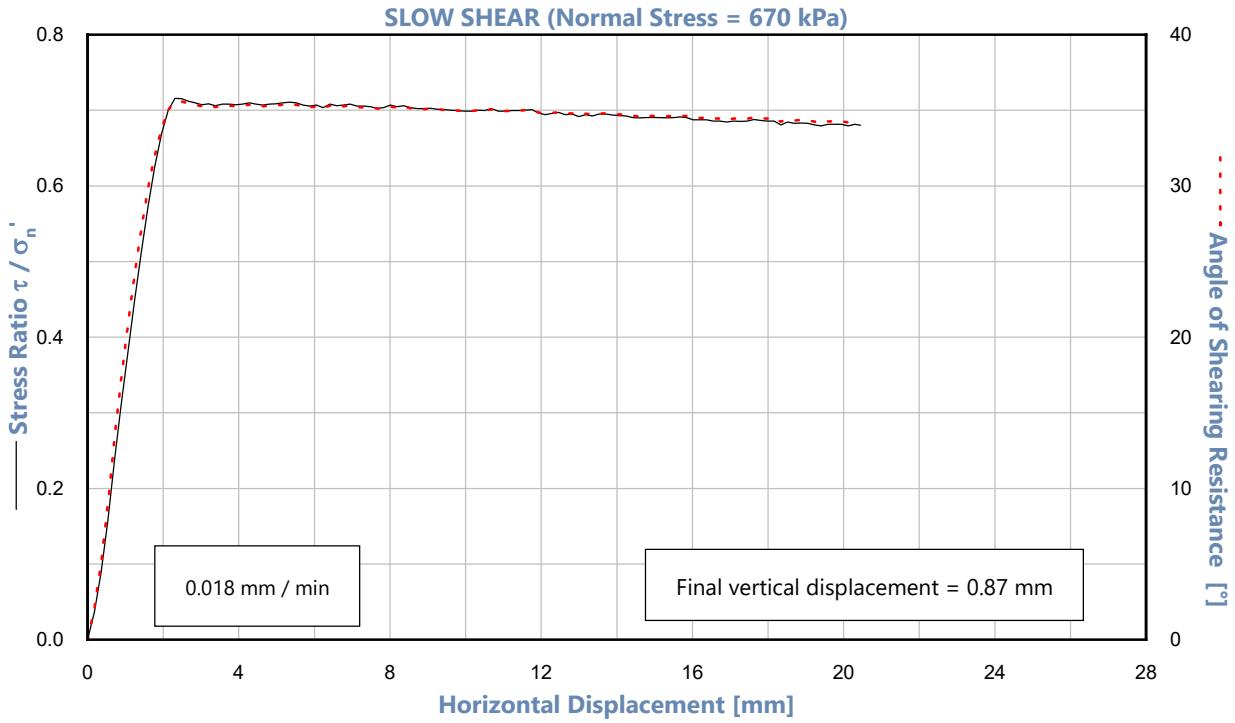
Checked By :
5.
Approved By





TEST IDENTIFICATION		
Borehole	NEPL7-BH01	Interface Roughness Before (µm) : 8.25
Sample	17BAGA	
Depth [m]	58.00	Interface Roughness After (µm) : --
Test number	RS04	

RING SHEAR TEST - SHEARING
SOIL / STEEL INTERFACE

Contents Appendix D.6: Results of Direct Shear Tests

List of Plates

Title	Plate
Results of Direct Shear Tests	D.6-1 to D.6-2

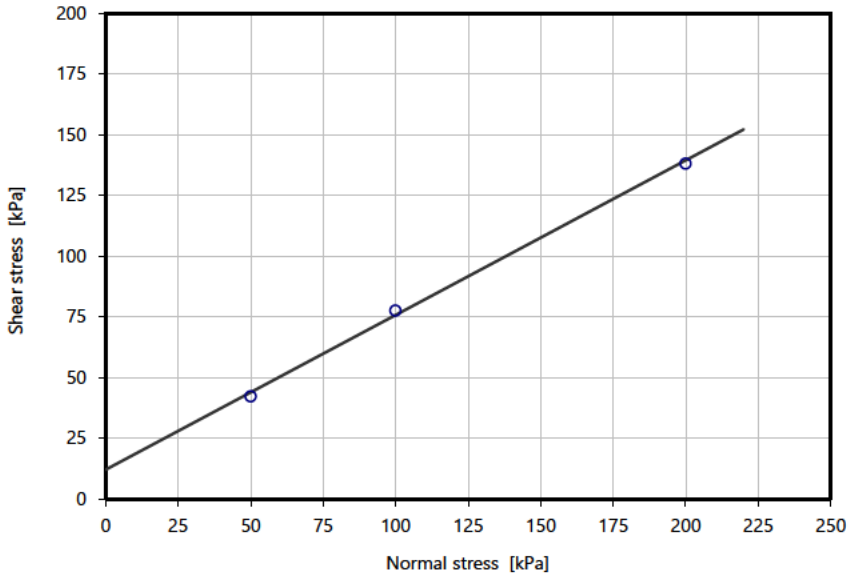
LABORATORY TEST CERTIFICATE

Determination of Shear Strength by Direct Shear (Small Shearbox Apparatus)

BS 1377-7:1990 Clause 4

1483

Project Reference	F225236	Location ID	NEPL7-BH01
Project Name	Neptune L7 -17	Depth Top [m]	0.20
Specimen Description	Brown slightly silty gravelly SAND	Sample Type	B
Specimen Reference		Specimen Depth [m]	Sample Reference 01BAGB



Shear Strength Parameters	Best Fit	Manual
c ' [kPa]	12	-
Ø ' [degrees]	32.5	-
c 'R [kPa]	not assessed	-
Ø 'R [degrees]	not assessed	-

Test number	1	2	3			
Particle Density [# indicates assumed value]	#2.65	#2.65	#2.65			
Height [mm]	24.5	24.3	25.0			
Bulk Density [Mg/m³]	1.56	1.56	1.56			
Initial Moisture Content [%]	10.3	10.3	10.3			
Dry Density [Mg/m³]	1.41	1.41	1.41			
Initial Voids Ratio	0.877	0.878	0.874			
Degree of Saturation [%]	31.2	31.1	31.2			
Consolidation						
Normal Stress [kPa]	50	100	200			
Change in Height [mm]	-0.880	-0.685	-1.088			
Post consolidation						
Voids Ratio	0.865	0.868	0.860			
Moisture content [%]	19.3	19.8	21.0			
Shearing stage(s)						
Number of Traverses (including peak run)	1	1	1			
Peak Values						
Rate of Displacement [mm/min]	0.012	0.012	0.012			
Relative Horizontal Displacement [mm]	2.26	1.81	2.39			
Shear Stress [kPa]	42.1	77.5	138.0			
Vertical Movement [mm]	-0.70	-0.55	-0.87			
Residual values						
Rate of Displacement [mm/min]						
Relative Horizontal Displacement [mm]						
Shear Stress [kPa]						
Vertical Movement [mm]						

Issue Date	04/05/2023	Certificate Reference		Authorised By	5.1.2.e
Client	Neptune			Authorised Date	28/04/2023 09:07
Remarks	Remoulded by tamping				

Fugro GeoServices Ltd. Unit 43, Number One Industrial Estate, Medomsley Road, Consett, DH8 6TW

Testing was performed at the Fugro GeoServices Ltd laboratory at the address shown above. Results relate only to the sample tested, having been authorised by persons qualified to do so. Opinions and interpretations are outside the scope of accreditation. Unless stated otherwise the sample was tested in the condition it was received at the laboratory.

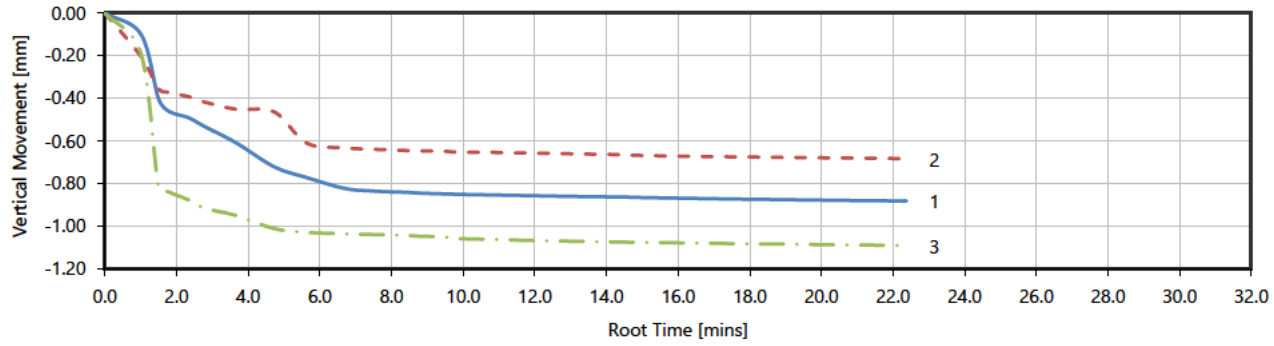
LABORATORY TEST CERTIFICATE

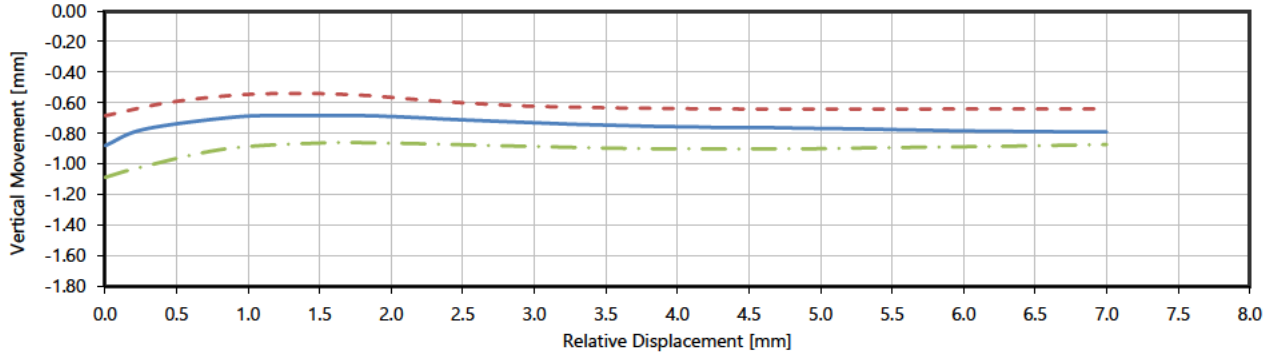
Determination of Shear Strength by Direct Shear (Small Shearbox Apparatus)

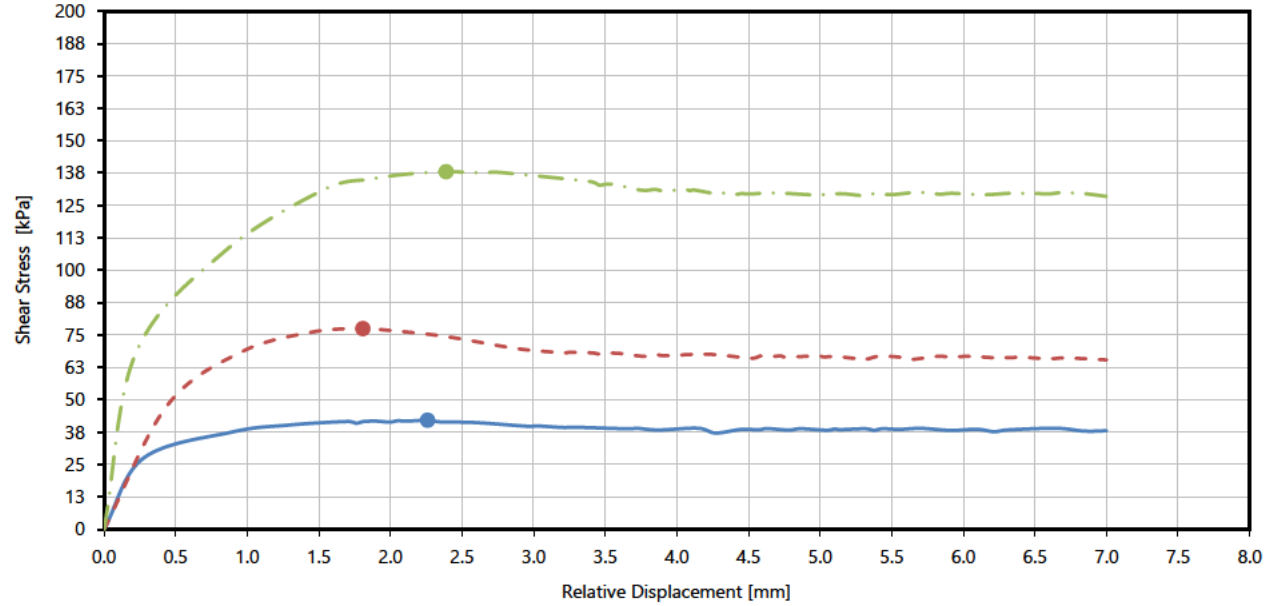
BS 1377-7:1990 Clause 4

1483

Project Reference	F225236			Location ID	NEPL7-BH01
Project Name	Neptune L7 -17			Depth Top [m]	0.20
Specimen Description	Brown slightly silty gravelly SAND			Sample Type	B
Specimen Reference		Specimen Depth [m]		Sample Reference	01BAGB







Issue Date	04/05/2023	Certificate Reference		Authorised By	5.1.2.e
Client	Neptune			Authorised Date	28/04/2023 09:07
Remarks	Remoulded by tamping				

Fugro GeoServices Ltd. Unit 43, Number One Industrial Estate, Medomsley Road, Consett, DH8 6TW

Testing was performed at the Fugro GeoServices Ltd laboratory at the address shown above. Results relate only to the sample tested, having been authorised by persons qualified to do so. Opinions and interpretations are outside the scope of accreditation. Unless stated otherwise the sample was tested in the condition it was received at the laboratory.

Appendix E

Descriptions of Methods and Practices

Contents Appendix E: Descriptions of Methods and Practices

List of Documents

Use of Geodata and Advice	FNLM-GEO-APP-077
Abbreviations	FNLM-GEO-APP-080
Symbols	FNLM-GEO-APP-017
Geotechnical Borehole	FNLM-GEO-APP-002
Geotechnical Log	FNLM-GEO-APP-078
Cone Penetration Test	FNLM-GEO-APP-001
Soil Description and Classification	FNLM-GEO-APP-005
Geotechnical Laboratory Tests	FNLM-GEO-APP-007
Geotechnical Parameter Values	FNLM-GEO-APP-012
Positioning Survey and Depth Measurement	FNLM-GEO-APP-029

Use of Geodata and Advice

Introduction

This document provides important information regarding the use of Fugro geodata, analyses and advice.

Site-specific acquisition of geodata can include metocean monitoring, geophysical seafloor mapping, subsurface mapping, logging of boreholes, in situ testing, laboratory testing of samples and monitoring of structures or elements of structures.

The cost of geodata acquisition, interpretation and monitoring is a small portion of the total cost of a development such as a construction project. By contrast, the costs of correcting a wrongly designed programme or mobilising alternative construction methods are often far greater than the cost of the original investigation for a site or structure.

Attention and adherence to the information presented in this document can reduce delays and cost overruns related to site-specific factors.

The focus of this document is on construction projects. This document also applies to information and advice related to asset integrity and decommissioning.

Requirements for Quality Geodata

Project quality management should follow quality principles for project management (e.g. ISO 9001:2015) and for general principles on reliability for structures (e.g. ISO 2394:2015). Project activities usually comprise part of specific phases of a construction project. The quality plan for the entire construction project should incorporate geodata input in every phase - from the feasibility planning stages to project completion. The parties involved should do the following.

- Provide complete and accurate information necessary to plan an appropriate site investigation.
- Describe the purpose(s), type(s) and construction methods of planned structures in detail.
- Provide the time, financial, personnel and other resources necessary for the planning, execution and follow-up of a site investigation programme.
- Understand the limitations and degree of accuracy inherent in geodata.
- Understand the limitations and degree of accuracy inherent in the advice based upon site investigation data.
- During all design and construction activities, be aware of the limitations of site investigation data and analyses/ advice, and use appropriate preventative measures.
- Incorporate all geodata input in the design, planning, construction and other activities involving the site and structures. Provide the entire (set of) document(s), including digital files where applicable, to parties involved in site selection, design and construction.
- Use the site investigation data and advice for only the structures, site and activities which were described to Fugro prior to and for the purpose of planning the site investigation or the programme of analysis and advice.

Authority, Time and Resources Necessary for Site Investigations

Adequate designation of authority and accountability for site-specific aspects of construction projects is necessary. This way, an appropriate investigation can be performed, and the use of the results by project design and construction professionals can be optimised.

Figure 1 illustrates the importance of the initial project phases for gathering adequate geodata for a project. The initial phases, when site investigation requirements are defined and resources are allocated, are represented by more than 50 % of the Quality Triangle (Figure 1). Decisions and actions made during these phases have a large impact of the outcome and thus the potential of the investigation to meet project requirements.

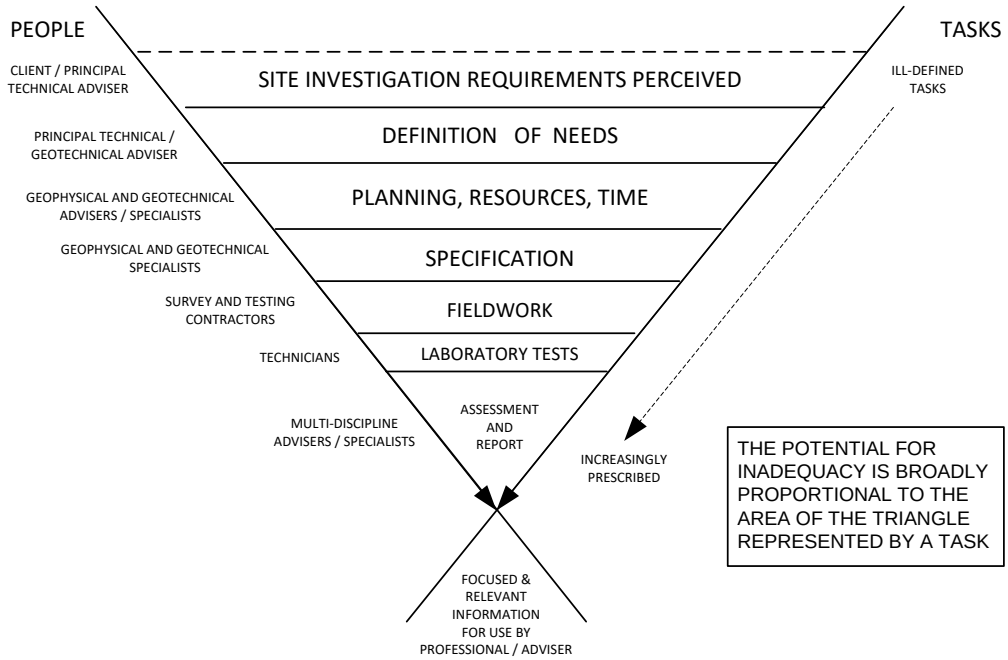


Figure 1: Quality of Site Investigation (adapted from SISG[1]).

[1] Site Investigation Steering Group SISG (1993). *Site investigation in construction 2: planning, procurement and quality management*. Thomas Telford.

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Data Acquisition and Monitoring Programmes

Site-specific investigations, such as geophysical and geotechnical investigations, are operations of discovery. Investigation should proceed in logical stages. Planning should allow operational adjustments deemed necessary by newly available information. This observational approach permits the development of a sound engineering strategy and reduces the risk of discovering unexpected (geo)hazards during or after construction.

Data Types and Limitations

1. Reliability of Supplied Information

Analysis and advice can involve the use of information and physical material that is publicly available or supplied by the client. Examples are geodetic data, geological maps, geophysical records, earthquake data, earlier geotechnical logs and soil samples. Fugro endeavours to identify potential anomalies but does not independently verify the accuracy or completeness of public or client-supplied information unless indicated otherwise. This information, therefore, can limit the accuracy of the geodata, analyses and advice.

2. Complexity of Ground Conditions

There are hazards associated with the ground. An adequate understanding of these hazards can help to minimize risks to a project and the site. The ground is a vital element of all structures which rest on or in the ground. Information about ground behaviour is necessary to achieve a safe and economical structure. Often less is known about the ground than for any other element of a structure.

3. Site Investigation - Spatial Coverage

Geophysical investigations typically provide information about ground conditions along survey track lines. Geotechnical investigations collect data at specific test locations. Interpretation of ground conditions away from survey track lines and test locations is a matter of extrapolation and judgement based on geological and geotechnical knowledge, as well as on experience. Nevertheless, actual conditions in untested areas may differ from predictions. For example, the interface between ground materials may be far more gradual or abrupt than indicated by the geodata. It is not realistic to expect a site investigation to reveal or anticipate every detail of ground conditions. Nevertheless, an investigation can reduce the residual risk associated with unforeseen conditions to a tolerable level. If ground problems do arise, it is important to have relevant expertise available to help reduce and mitigate safety and financial risks.

4. Role of Judgement and Opinion In Analysis and Advice

Analysis and advice that involve geodata are less exact than most other design disciplines. Extensive judgement and opinion are often required. Therefore, geodata, analyses and advice may contain definitive statements that identify where the responsibility of Fugro begins and ends. These are not exculpatory clauses designed to transfer liabilities to another party, but they are statements that can help all parties involved to recognise their individual responsibilities and take appropriate actions.

Complete Information should be Available to all Parties Involved

To prevent costly construction problems, construction contractors should have access to the best available information. They should have access to the complete original (set of) documents including digital files where applicable, to prevent or minimize any misinterpretation of site conditions and advice. To prevent errors or omissions that could lead to misinterpretation, geophysical sections, geotechnical logs and illustrations should not be redrawn or reprocessed, and users of geodata and advice should confer with the authors when applying the geodata and/or advice. A cloud-hosted, web-based geodata engagement platform can facilitate information access throughout the life cycle of an asset.

Information is Project-Specific

Fugro's investigative programmes, analyses and advice are designed and conducted specifically for the client described project and conditions. Thus the geodata, analyses and advice present information for a unique construction project. Project-specific factors for a structure include but are not limited to:

- Location;
- Size and configuration of structure;
- Type and purpose or use of structure;
- Other facilities or structures in the area.

Any factor that changes subsequent to the preparation of the geodata, analyses and advice may affect its applicability. A specialised review of the impact of changes would be necessary. Fugro is not responsible for conditions which develop after change of any factor in site investigation programming, development or structure.

For purposes or parties other than the original project or client, the geodata, analyses and advice may not be adequate and should not be used.

Changes in Site Conditions Affect the Accuracy/Suitability of the Data

Ground is complex and can be changed by natural phenomena such as earthquakes, floods, seabed scour and groundwater fluctuations. Construction operations at or near the site can also change ground conditions. The geodata, analyses and advice consider conditions at the time of investigation. Construction decisions should consider any changes in site conditions, regulatory provisions, technology or economic conditions subsequent to the investigation. Geodata, analyses and advice can become inaccurate or unreliable upon any passage of time, and a specialist should be consulted regarding the adequacy of the geodata, analyses and advice for use.

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Abbreviations

I – General

1D	one-dimensional
2D	two-dimensional
3D	three-dimensional
AI	artificial intelligence
FEED	front-end engineering design
GIS	geographical information system
HAZID	hazard identification
HIRA	hazard identification and risk assessment
HSE	health, safety and environment
HSSE	health, safety, security and environment
ML	machine learning
PDF	portable document format
QHSSE	quality, health, safety, security and environment
SIM	structural integrity management

II – Standards

API	American Petroleum Institute
ASTM	American Society for Testing and Materials
BS	British standard
EN	Europäische norm (European standard)
ISO	International Organization for Standardization

III – Statistics

CDF	cumulative distribution function
COV	coefficient of variation
PDF	probability density function
SD	standard deviation

IV – Geodetics

BGL	below ground level
BSF	below seafloor
CD	chart datum
CM	central meridian
DGPS	Differential Global Positioning System
CRS	coordinate reference system
E	east
ED	European Datum
ETRS	European terrestrial reference system
GNSS	Global Navigation Satellite System
GPS	Global Positioning System
GRS	Geodetic Reference System
INS	inertial navigation system
KP	kilometre point
LAT	Lowest Astronomical Tide
MSL	Mean Sea Level
N	north
S	south
SWL	still water level
TM	Transverse Mercator
USBL	ultra short baseline
UTC	Coordinated Universal Time
UTM	Universal Transverse Mercator
W	west
WGS	World Geodetic System

V – Survey Platforms

ASV	autonomous surface vehicle
ATV	all-terrain vehicle
AUV	autonomous underwater vehicle
MV	motor vessel
ROV	remotely operated vehicle
SV	sailing vessel

VI – Site Investigation

ABI	acoustic borehole imager
BC	box core(r)
BH	borehole
BGL	borehole geophysical logging
BPT	ball penetration test
CAL	caliper tool
CDP	common depth point
CMP	common mid-point
CPT	cone penetration test
CPTU	piezocone penetration test (or PCPT)
CTD	conductivity temperature depth
DMR	downhole magnetic resonance
FFP	free-fall penetration test
FLPC	Fugro large piston core(r)
GC	gravity core(r)
GR	natural gamma radiation
GS	grab sample(r)
HF-CPTU	CPTU with heat flow test
HF-T-CPT	T-CPT with heat flow test
HF-NPT	thermal needle probe test
HRS	high resolution seismic reflection
ID	inner diameter
LDPC	large diameter piston core(r)
LGPC	large gravity piston core(r)
MAG	magnetometer
MBES	multibeam echosounder
MBPT	miniature ball penetration test
MCS	multichannel seismic reflection
MTPT	miniature T-bar penetration test
OBI	optical borehole imager
OD	outer diameter
PC	piston core(r)
PPDT	pore pressure dissipation test
PSSL	P and S suspension logger
RC	rotary core(r)
SBES	single beam echosounder
SBF	seabed frame
SBP	sub-bottom profiler, seismic reflection
SCPT	seismic cone penetration test
SCS	single channel seismic reflection
SGR	spectral gamma radiation
SIR	strong impedance reflector
SSS	side scan sonar
STACOR®	stationary piston gravity core(r)
SVP	sound velocity profiler
T-CPT	temperature cone penetration test
TPT	T-bar penetration test
TWTT	two-way travel time, seismic reflection
UHR	ultra high resolution
UHRS	ultra high resolution seismic reflection
UUHR	ultra ultra high resolution
VC	vibrocore(r)
VSP	vertical seismic profiler
VST	vane shear test
WISON®	wireline sounding tool
WIP	wireline push sampler

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VII – Site Characterisation

BP	before present
DSHA	deterministic seismic hazard assessment
DTM	digital terrain model
Fm	geological formation
LGM	last glacial maximum
Mb	geological formation member
MDAC	methane-derived authigenic carbonate
MTD	mass transport deposit
PSHA	probabilistic seismic hazard assessment
SBT	soil behaviour type
SGHA	shallow gas hazard assessment
SSDM	seabed survey data model
UXO	unexploded ordnance

VIII – Geotechnical Analysis

ALS	accidental limit state
ASD	allowable stress design
FLS	fatigue limit state
FOS	factor of safety
LSD	limit state design
PFD	partial factor design
SCR	steel catenary riser
SLS	serviceability limit state
SRD	soil resistance to driving
UHC	ultimate holding capacity
ULS	ultimate limit state
VLA	vertically loaded anchor
WSD	working stress design
WTG	wind turbine generator

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Symbols

Symbol	Unit	Quantity
I - General		
A	m^2	Area
L	m	Length
B	m	Width
D	m	Diameter
U	-	Uncertainty of parameter value
V	m^3	Volume
W	kN	Weight
a	m/s^2	Acceleration
d	m	Depth
f	Hz	Frequency ($= 1/T$)
g	m/s^2	Acceleration due to gravity ($g = 9.81$ m/s^2)
h	m	Height or thickness
i	-	Inclination
m	kg	Mass
T	s	Period (time taken to complete one cycle of an oscillation)
t	s	Time
t	a	Time, mean Julian year
v	m/s	Velocity
z	m	Penetration or depth below reference level (usually ground surface)
$\dot{\varepsilon}$	s^{-1}	Rate of strain (length)
ρ	kg/m^3	Density
π	-	Mathematical constant ($= 3.14159$)
e	-	Base of natural logarithm ($= 2.71828$)
ln	-	Natural logarithm
log	-	Logarithm base 10
II - Stress and Strain		
P_a	kPa	Atmospheric pressure
u	MPa	Pore pressure
u_o	MPa	Hydrostatic pore pressure relative to seafloor or phreatic surface
u_f	MPa	Pore pressure at failure
Δu	MPa	Change in pore pressure or excess pore pressure
σ	kPa	Total stress
σ'	kPa	Effective stress
τ	kPa	Shear stress
τ_{peak}	kPa	Peak shear stress
$\sigma_1, \sigma_2, \sigma_3$	kPa	Principal stresses
σ_h	kPa	Total horizontal stress
$\Delta\sigma_h$	kPa	Change in total horizontal stress
σ_v	kPa	Total vertical stress
$\Delta\sigma_v$	kPa	Change in total vertical stress
σ_{h0}	kPa	Total in situ horizontal stress relative to ground surface or phreatic surface
σ'_{h0}	kPa	Effective in situ horizontal stress
σ_{v0}	kPa	Total in situ vertical stress relative to ground surface or phreatic surface
σ'_{v0}	kPa	Effective in situ vertical stress (or $p'_{0)}$
σ'_h	kPa	Effective horizontal stress
σ'_v	kPa	Effective vertical stress
σ'_r	kPa	Effective radial stress
σ'_a	kPa	Effective axial stress
r_u	-	Pore pressure ratio [$= u/\sigma_{v0}$]
p'	kPa	Mean effective stress [$= (\sigma'_1 + \sigma'_2 + \sigma'_3)/3$]
q	kPa	Principal deviator stress [$= \sigma'_1 - \sigma'_3$] or [$= \sigma_1 - \sigma_3$]
q_{ref}	kPa	Reference principal deviator stress
s'	kPa	Mean effective stress in $s' - t$ space [$= (\sigma'_1 + \sigma'_3)/2$]
t	kPa	Shear stress in $s' - t$ space [$= (\sigma'_1 - \sigma'_3)/2$] or [$= (\sigma_1 - \sigma_3)/2$]

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Symbol	Unit	Quantity
ε	-	Linear strain
$\varepsilon_1, \varepsilon_2, \varepsilon_3$	-	Principal strains
ε_v	-	Vertical strain (or volumetric strain)
ε_{vol}	-	Volumetric strain
γ	-	Shear strain
γ_c	-	Shear strain at maximum shear stress
ν	-	Poisson's ratio
ν_u	-	Poisson's ratio for undrained stress change
ν_d	-	Poisson's ratio for drained stress change
E	MPa	Modulus of linear deformation (Young's modulus)
E_{max}	MPa	Modulus of linear deformation at small strain
E_u	MPa	Modulus of linear deformation (Young's modulus for undrained stress change)
E_d	MPa	Modulus of linear deformation (Young's modulus for drained stress change)
G	MPa	Modulus of shear deformation (shear modulus)
G_{max}	MPa	Small strain shear modulus (shear modulus at small strain)
I_r	-	Rigidity index [$= G/\tau_{max}$ or G/s_u]
K	MPa	Modulus of compressibility (bulk modulus)
M	MPa	Constrained modulus [$= 1/m_v$]
M_{max}	MPa	Constrained modulus at small strain
μ	-	Coefficient of friction
η	kPa.s	Coefficient of viscosity

III - Physical Characteristics of Ground

(a) Density and Unit Weights

γ	kN/m^3	Unit weight of ground (or bulk unit weight or total unit weight)
γ_d	kN/m^3	Unit weight of dry ground
γ_s	kN/m^3	Unit weight of solid particles
γ_w	kN/m^3	Unit weight of water
γ_{pf}	kN/m^3	Unit weight of pore fluid
γ_{dmin}	kN/m^3	Minimum index (dry) unit weight
γ_{dmax}	kN/m^3	Maximum index (dry) unit weight
γ'	kN/m^3	Unit weight of submerged ground (or γ_{sub})
ρ	Mg/m^3 [= t/m^3]	Density of ground (or bulk density)
ρ_d	Mg/m^3 [= t/m^3]	Density of dry ground
ρ_s	Mg/m^3 [= t/m^3]	Density of solid particles (or G_s)
ρ_w	Mg/m^3 [= t/m^3]	Density of water
D_r	-, %	Relative density [$= I_D = \gamma_{dmax}(\gamma_d - \gamma_{dmin})/\gamma_d(\gamma_{dmax} - \gamma_{dmin})$ = $(e_{max} - e)/(e_{max} - e_{min})$]
ν	-	Specific volume [$= 1 + e$]
e	-	Void ratio
e_0	-	Initial void ratio
$e_{\sigma'_{v0}}$	-	Void ratio at σ'_{v0} (or e_0)
e_{max}	-	Maximum index void ratio
e_{min}	-	Minimum index void ratio
G_s	-	Specific gravity of solid particles
I_D	-, %	Density index [$= D_r$]
R_D	-, %	Dry density ratio [$= \gamma_d/\gamma_{dmax}$]
n	-, %	Porosity
w	%	Water content
S_r	%	Degree of saturation
r	-, g/kg	Salinity of pore fluid [= ratio of mass of salt to mass of pore fluid]
R	g/l	Salinity of fluid [= ratio of mass of salt to volume of distilled water]
s	g/l	Salinity of fluid [= ratio of mass of salt to volume of fluid]
S	g/kg	Salinity of seawater [= ratio of mass of salt to mass of seawater]

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Symbol	Unit	Quantity
(b) Consistency		
w_L	%	Liquid limit
w_P	%	Plastic limit
I_P	%	Plasticity index [$= w_L - w_P$]
I_L	%	Liquidity index [$= (w - w_P)/(w_L - w_P)$]
I_C	%	Consistency index [$= (w_L - w)/(w_L - w_P)$]
A	-, %	Activity [= ratio of I_P to percentage by weight of clay-size particles]
(c) Particle Size		
D	mm	Particle diameter
D_n	mm	Particle diameter, where n% of the dry mass of ground has a smaller particle diameter
C_u	-	Uniformity coefficient [$= D_{60}/D_{10}$]
C_c	-	Curvature coefficient [$= (D_{30})^2/D_{10}D_{60}$]
FC_D	mm	Fines content, as % of the dry mass of ground having particle diameters smaller than D in mm (e.g. $FC_{0.075}$ for ASTM methods and $FC_{0.063}$ for ISO methods)
(d) Acoustic Properties		
S_{vh}	-	S-wave propagating in the vertical direction with particle motion in the horizontal direction
S_{hh}	-	S-wave propagating in the horizontal direction with particle motion in the horizontal direction
S_{hv}	-	S-wave propagating in the horizontal direction with particle motion in the vertical direction
v_p	m/s	P-wave velocity (compression wave velocity)
v_s	m/s	S-wave velocity (shear wave velocity)
v_{s1}	m/s	S-wave velocity normalised to 100 kPa in situ vertical stress
v_{vh}	m/s	S-wave velocity, vertically (v) propagated, horizontally (h) polarised
(e) Hydraulic Properties		
K	m/s	Hydraulic conductivity (or k where permeating fluid consists of water)
k	m/s	Coefficient of permeability
k_v	m/s	Coefficient of vertical permeability
k_h	m/s	Coefficient of horizontal permeability
i	-	Hydraulic gradient
(f) Thermal Properties		
T	K, °C	Temperature
k	W/(m·K)	Thermal conductivity
a_L	1/°C	Thermal expansion coefficient (linear)
α	m^2/s	Thermal diffusion coefficient
C	MJ/m^3K	Volumetric heat capacity
(g) Electrical Properties		
ρ	Ωm	Electrical resistivity
K	S/m	Electrical conductivity
(h) Magnetic Properties		
B	T	Magnetic flux density (or magnetic induction)
(i) Radioactive Properties		
γ	CPS	Natural gamma ray

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IV - Mechanical Characteristics of Ground

(a) Cone Penetration Test (CPT)		
q_c	MPa	Cone resistance
q_{c1}	MPa	Cone resistance normalised to 100 kPa effective in situ vertical stress
f_s	MPa	Sleeve friction

Symbol	Unit	Quantity
f_t	MPa	Sleeve friction corrected for pore pressures acting on the end areas of the friction sleeve
R_f	%	Ratio of sleeve friction to cone resistance
R_{ft}	%	Ratio of sleeve friction to corrected cone resistance (f_s/q_t or f_t/q_t)
u_1	MPa	Pore pressure at the face of the cone
u_2	MPa	Pore pressure at the cylindrical extension above the base of the cone or in the gap between the friction sleeve and the cone
$u_2{}^*$	MPa	Pore pressure u_2, but derived rather than measured
u_3	MPa	Pore pressure immediately above the friction sleeve or in the gap above the friction sleeve
K	-	Adjustment factor for ratio of pore pressure at u_1 to u_2 location
q_n	MPa	Net cone resistance
q_t	MPa	Corrected cone resistance (or total cone resistance)
B_q	-	Pore pressure ratio
Q_t	-	Normalised cone resistance [$= q_n/\sigma'_{v0}$]
Q_{tn}	-	Normalised cone resistance with variable stress exponent
F_r	%	Normalised friction ratio [$= f_t/q_n$]
U_2	-	Normalised excess pore pressure [$= (u_2 - u_0)/\sigma'_{v0}$]
CD	-	Contractive-dilative boundary (for soil behaviour type)
I_B	-	Soil behaviour type index (for Q_{tn} and F_r)
I_c	-	Soil behaviour type index (for Q_{tn} and F_r)
I_{SBT}	-	Soil behaviour type index (for q_c and R_f)
N_c	-	Cone factor between q_c and s_u
N_{kt}	-	Cone factor between q_n and s_u (or N_k)
$N_{\Delta u}$	-	Pore pressure factor between $u_2 - u_0$ and s_u

(b) Standard Penetration Test (SPT)

N	Blows/0.3 m	SPT blow count
N_{60}	Blows/0.3 m	SPT blow count normalised to 60 % energy
$N_{1,60}$	Blows/0.3 m	SPT blow count normalised to 60 % energy and to 100 kPa effective in situ vertical stress

(c) Strength and Stiffness of Soil – Static

s_u	kPa	Undrained shear strength (or c_u)
s_{uC}	kPa	Undrained shear strength in laboratory triaxial compression (or c_{uC})
s_{uD}	kPa	Undrained shear strength in laboratory direct simple shear (or c_{uD})
s_{uE}	kPa	Undrained shear strength in laboratory triaxial extension (or c_{uE})
$s_{u;ref}$	kPa	Reference undrained shear strength
s_u/σ'_{v0}	-	Undrained strength ratio
$s_{u;a1}/s_{u;a2}$	-	Strength anisotroppy ratio – undrained shear strength
κ	kPa/m	Rate of increase of undrained shear strength with depth (linear)
c'	kPa	Effective cohesion intercept
φ'	°(deg)	Effective angle of internal friction (or ϕ')
φ'_{cv}	°(deg)	Effective angle of internal friction at large strain (or ϕ'_{cv})
φ'_R	°(deg)	Effective angle of internal friction at residual shear conditions (or ϕ'_R)
ψ	°(deg)	Angle of dilation (or dilatancy angle)
ε_{50}	%	External axial strain at half the maximum deviator stress (or ε_c)
ε_c	%	External axial strain at the maximum deviator stress
E_{50}	MPa	Secant Young's modulus at half the maximum deviator stress
$s_{u;r}$	kPa	Undrained shear strength of remoulded soil
$s_{u;ar}$	kPa	Undrained shear strength of aged remoulded soil
s_R	kPa	Undrained residual shear strength
S_t	-	Sensitivity [$= s_u/s_{u;r}$ or s_u/s_R]
T_x	-	Thixotropy strength ratio $[T_x(t) = s_{u;ar}(t)/s_{u;r}]$
M	-	Gradient of critical state line when projected onto a constant volume plane
A	-	Pore pressure coefficient for anisotropic pressure increment
B	-	Pore pressure coefficient for isotropic pressure increment

(d) Strength and Stiffness of Soil – Cyclic and Dynamic

N	-	Number of cycles (or cycle number)
N_f	-	Number of cycles to soil failure or final number of cycles
N_{eq}	-	Equivalent number of cycles
τ_0	kPa	Initial shear stress
τ_{av}	kPa	Average shear stress or constant shear stress (or τ_a)

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Symbol	Unit	Quantity
τ_{cy}	kPa	Cyclic shear stress amplitude [= $(\tau_{max} - \tau_{min})/2$]
$\tau_{cy,f}$	kPa	Cyclic shear strength at a specified failure criterion [= $(\tau_{av} + \tau_{cy})_f$]
τ_{max}	kPa	Maximum shear stress
τ_{min}	kPa	Minimum shear stress
γ_{av}	%	Average shear strain (or γ_a) [= $(\gamma_{max} + \gamma_{min})/2$]
γ_{cy}	%	Cyclic shear strain amplitude [= $(\gamma_{max} - \gamma_{min})/2$]
γ_{max}	%	Maximum shear strain
γ_{min}	%	Minimum shear strain
γ_p	%	Permanent shear strain
σ'_{ac}	kPa	Effective axial consolidation stress
σ'_{rc}	kPa	Effective radial consolidation stress
σ'_{ref}	kPa	Reference effective stress
q_{cy}	kPa	Cyclic deviator stress amplitude [= $(q_{max} - q_{min})/2$]
q_{max}	kPa	Maximum deviator stress
q_{min}	kPa	Minimum deviator stress
q_{av}	kPa	Average deviator stress [= $(q_{max} + q_{min})/2$]
ε_{cy}	%	Cyclic axial strain (or cyclic vertical strain) amplitude [= $(\varepsilon_{max} - \varepsilon_{min})/2$]
ε_{max}	%	Maximum axial strain (or maximum vertical strain)
ε_{min}	%	Minimum axial strain (or minimum vertical strain)
ε_{av}	%	Average axial strain (or average vertical strain) [= $(\varepsilon_{max} + \varepsilon_{min})/2$]
ε_a	%	External axial strain at N_f (or external vertical strain at N_f)
E_{ext}	MPa	Young's modulus derived from loop stiffness and external axial strain
E_{loc}	MPa	Young's modulus derived from loop stiffness and local axial strain
u_a	kPa	Average pore pressure
u_{cy}	kPa	Cyclic pore pressure amplitude [= $(u_{max} - u_{min})/2$]
u_{max}	kPa	Maximum pore pressure
u_{min}	kPa	Minimum pore pressure
u_p	kPa	Permanent pore pressure
λ	-, %	Damping ratio of ground (or D)
λ_{ext}	%	Damping ratio derived from external axial strain
λ_{loc}	%	Damping ratio derived from local axial strain

(e) Strength of Rock

σ_c	MPa	Uni-axial compressive strength
$I_{s(50)}$	MPa	Point load strength index
NPR	N/mm	Needle point resistance

(f) Consolidation (One Dimensional)

σ'_{hc}	kPa	Effective horizontal consolidation stress
σ'_{vc}	kPa	Effective vertical consolidation stress
σ'_p	kPa	Effective preconsolidation stress (or effective vertical yield stress in situ)
σ^*_{ve}	kPa	Effective vertical stress on ICL at e_0
σ'_{vy}	kPa	Effective vertical yield stress in situ (or effective preconsolidation stress)
C_c	-	Compression index
C^*_c	-	Intrinsic compression index [= $e^*_{100} - e^*_{1000}$]
C_s	-	Swelling index (or re-compression)
CR	-	Primary compression ratio [= $C_c/(1 + e_0)$]
RR	-	Recompression ratio [= $C_s/(1 + e_0)$]
e_L	-	Void ratio at liquid limit w_L
e^*_{100}	-	Void ratio at σ'_v = 100 kPa during one-dimensional intrinsic compression
e^*_{1000}	-	Void ratio at σ'_v = 1000 kPa during one-dimensional intrinsic compression
C_α	-	Coefficient of secondary compression (primary compression)
$C_{\alpha s}$	-	Coefficient of secondary compression (swelling/re-compression)
c_v	m²/s	Coefficient of consolidation
H	m	Drainage path length
ICL	-	Intrinsic compression line (Burland, 1990)
I_v	-	Void index [= $(e_0 - e^*_{100})/ C^*_c$]
m_v	m²/MN	Coefficient of volume compressibility
M	MPa	Constrained modulus [= $1/m_v$]
p	kPa	Vertical pressure
OCR	-	Overconsolidation ratio [= σ'_p/σ'_{v0}] (or yield stress ratio)

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Symbol	Unit	Quantity
SCC	-	Sedimentation compression curve
SCL	-	Sedimentation compression line (Burland, 1990)
S_σ	-	Stress sensitivity [$= \sigma'_{vy}/\sigma^*_{ve}$]
YSR	-	Yield stress ratio [$= \sigma'_{vy}/\sigma'_{v0}$] (or overconsolidation ratio)

V - Geotechnical Design

(a) Partial Factors

γ_d	-	Factor related to model uncertainty or other circumstances
γ_f	-	Partial action factor (load factor)
γ_m	-	Partial material factor (partial safety factor)
γ_R	-	Partial resistance factor (partial safety factor)

(b) Seismicity

a_g	m/s^2	Effective peak ground acceleration (design ground acceleration)
d_g	m	Peak ground displacement
α	-	Acceleration ratio [$= a_g/g$]
τ_c	kPa	Seismic shear stress

(c) Compaction

ρ_{dmax}	Mg/m^3 [= t/m^3]	Maximum dry density
ρ_{max}	Mg/m^3 [= t/m^3]	Maximum density
w_{opt}	%	Optimum moisture content

(d) Earth Pressure

δ	°(deg)	Angle of interface friction (between ground and foundation)
K	-	Coefficient of lateral earth pressure
K_a	-	Coefficient of active earth pressure
K_{ac}	-	Coefficient of active earth pressure for total stress analysis
K_p	-	Coefficient of passive earth pressure
K_{pc}	-	Coefficient of passive earth pressure for total stress analysis
K_0	-	Coefficient of earth pressure at rest
K_{0nc}	-	K_0 for normally consolidated soil
K_{0oc}	-	K_0 for overconsolidated soil

(e) Foundations

A	m^2	Total foundation area
A'	m^2	Effective foundation area
B'	m	Effective width of foundation
E_s	MN/m^3	Modulus of subgrade reaction
k	MPa/m	Rate of change of modulus of subgrade reaction E_s with depth z
L'	m	Effective length of foundation
H	MN	Horizontal external force or action
V	MN	Vertical external force or action
M	MN.m	External moment
T	MN.m	External torsion moment
Q	MN	Total vertical resistance of a foundation/pile
Q_p	MN	End bearing of pile
Q_s	MN	Shaft resistance of pile
q_p	MPa	Unit end bearing
q_{lim}	MPa	Limit unit end bearing
f	kPa	Unit skin friction (or q_s)
f_{lim}	kPa	Limit unit skin friction
p	MN/m	Lateral resistance per unit length of pile
p_{lim}	MN/m	Limit lateral resistance per unit length of pile
s	m	Settlement
t	MN/m	Skin friction per unit length of pile
y	mm	Lateral pile deflection

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Symbol	Unit	Quantity
z	mm	Axial pile displacement
α	-	Adhesion factor between ground and foundation (= f/s_u)
β	-	Adhesion factor between ground and foundation (= f/σ'_v or f/σ'_{v0})
δ	°(deg)	Angle of interface friction (between ground and foundation)
δ_{cv}	°(deg)	Constant volume or critical-state angle of interface friction (between ground and foundation)
N_c, N_q, N_γ	-	Bearing capacity factors
K_c, K_q, K_γ	-	Bearing capacity correction factors for inclined forces or actions, foundation shape and depth of embedment
i_c, i_q, i_γ	-	Bearing capacity correction factors for external force inclined from vertical shape
S_c, S_q, S_γ	-	Bearing capacity correction factors for foundation shape
d_c, d_q, d_γ	-	Bearing capacity correction factors for foundation embedment

Signs:
- A 'prime' applies to effective stress.
- A 'bar' above a symbol relates to average properties.
- A 'dot' above a symbol denotes derivative with respect to time.
- The prefix 'Δ' denotes an increment or a change.
- A 'star' after a symbol denotes value corrected for pore fluid salinity.

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Geotechnical Borehole

Introduction

This document describes borehole activities for a geotechnical project. The activities comprise borehole drilling and, optionally (1) borehole geophysical logging, (2) in situ testing in borehole and/or (3) sampling and sample handling.

The common drilling techniques for onshore investigations are:

- Open hole drilling: a drilling method whereby all material within the diameter of the borehole is cut, such as open hole rotary drilling, cable percussion drilling and auger drilling;
- Open hole rotary drilling: an open hole drilling method whereby ground at the bottom of the borehole is cut by a drill bit rotated on the bottom of a borehole, and drill fluid is pumped down to the drill bit through the hollow drill pipe;
- Cable percussion drilling: an open hole drilling method whereby ground at the bottom of the borehole is broken up by percussive action of a bailer, clay cutter or chisel, and brought to the surface by the bailer or clay cutter;
- Auger drilling: an open hole drilling method whereby ground at the bottom of the borehole is cut and brought to the surface by auger flights;
- Core drilling: a rotary drilling method that cuts out cylindrical ground samples.

The common drilling techniques for marine investigations are open hole rotary drilling and core drilling. Core drilling can be combined with the following deployments modes of Figure 1: (b), (e) and (f). Offshore core drilling can also be achieved by piggyback operations, whereby the drill pipe for open hole rotary drilling acts as drill casing and as support for the drill rig. Piggyback operations can be combined with the following deployments modes of Figure 1: (b), (d) and (e).

ISO (2014) recognises the following deployment modes: (1) non-drilling, (2) vessel drilling and (3) seafloor drilling. Fig. 1(a) illustrates non-drilling, i.e. deployment of an in situ testing tool or sampling tool from seafloor, in single stroke with no drill-out. Strictly, non-drilling also applies to Fig. 1(c), but ISO considers this jack-up mode as vessel drilling. Fig. 1(e) illustrates a hybrid system, in-between vessel drilling and seafloor drilling. Fugro operates this hybrid system under SEADEVIL®. Drilling control is achieved by means of a combination of a drill rig at vessel deck and a vertical control system at seafloor. Fig. 1(f) illustrates seafloor drilling. Fugro operates this system under SeaFloor Drill SFD. Looijen and Peuchen (2017) provide a comparison of vessel drilling and seafloor drilling technologies, including the SEADEVIL®.

An optional activity for rotary drilling is the recording of drilling parameters by automated logging.

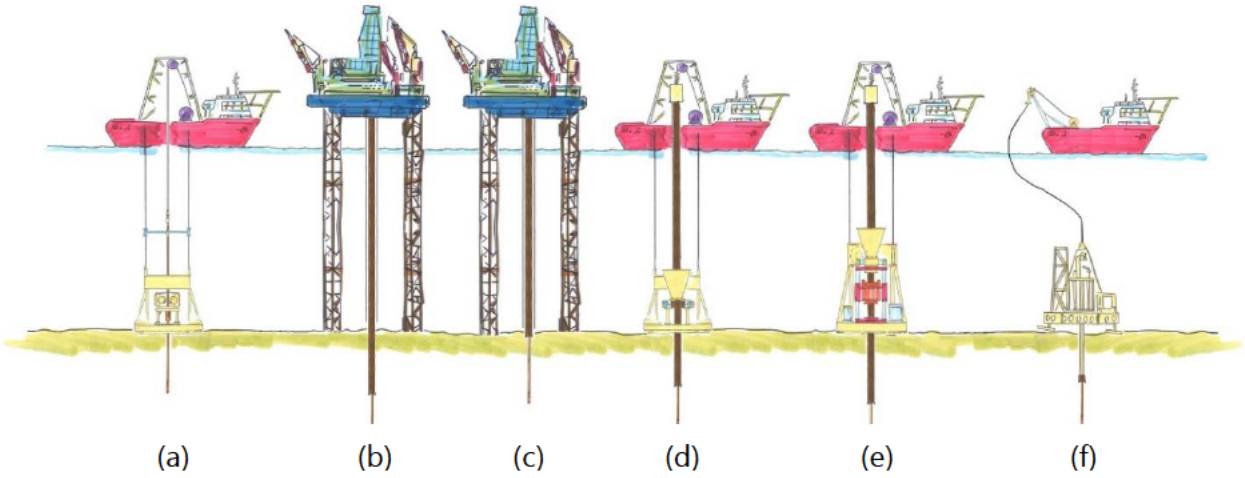

Figure 1: Deployment modes for marine soil investigation, in combination with cone penetration testing

Borehole geophysical logging tools are available for in-pipe measurements and for open hole measurements including tools such as the P and S suspension logger and the acoustic televiewer.

A wide range of in situ tests is available for boreholes. Examples are the pore pressure dissipation test (PPDT), vane shear test (VST) and cone penetration test (CPT).

This document describes borehole geophysical logging and in situ tests as an integral part of borehole activities, but gives no details. Separate descriptions apply, where appropriate.

The common borehole sampling techniques are drive sampling and/or push sampling of an open-tube sampler, and push sampling in case of a piston sampler. Sampling of cuttings from drilling may be feasible for some types of drilling techniques.

Borehole activities are based on ISO, CEN, BSI and ASTM standards.

Drilling Apparatus

General

Descriptions of common borehole drilling apparatus are as follows:

- Drilling equipment: any equipment that provides a suitably clean open hole before insertion of downhole sampling and/or testing apparatus and ensures that sampling and/or testing is performed in undisturbed ground;
- Drill rig: machine capable of providing:
  - Rotation, feed and retraction to drill pipe, casing and/or auger;
  - Drill fluid pumping capacity, as required;
  - Sampler or test apparatus insertion;
- Drill casing: cylindrical pipe with one or more of the following purposes:
  - To support the sides of a borehole;
  - To support drill pipe above ground surface in case of over-water drilling;
  - To promote return of drilling fluid;

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- Drill pipe: cylindrical pipe connecting drill rig and drill bit;
- Drill collar: thick-walled drill pipe providing self-weight thrust for the drill bit;
- Drill bit: device attached to drill pipe and used as a cutting tool to drill into the ground;
- Core bit: device attached to a rotary core sampler and used as a cutting tool to drill into the ground;
- Core catcher: device that assists retention of core in the core barrel;
- Core box: box with longitudinal separators for the protection and storage of (core) sample;
- Bottom hole assembly: lower section of offshore drill pipe and drill bit, shaped to permit latching of downhole in situ testing and sampling apparatus.

Coring

Core drilling is a ground investigation technique comprising simultaneous drilling and sampling. Descriptions of apparatus are as follows:

- Single tube core barrel: hollow steel tube with a head at the upper end threaded for drill pipe, and a threaded connection for the core bit at the lower end;
- Double tube core barrel: assembly of two concentric steel tubes joined at the upper end by means of a swivel arranged to permit rotation of the outer tube without causing rotation of the inner tube; the upper end of the outer tube is threaded for drill pipe and the lower end is threaded for the core bit;
- Double tube core barrel with retrievable inner tube: double tube core barrel that permits retrieval of the core-laden inner tube assembly to the surface through matching drill pipe without the need for withdrawal of the drill pipe.

Offshore Operations

Offshore drilling can require additional apparatus, particularly when drilling from a vessel:

- Seafloor template (or seabed reaction frame): seafloor-based apparatus capable of providing one or more of the following:
  - Improved horizontal and vertical control of the drill pipe;
  - Re-entry of a borehole by drill pipe after earlier retraction;
  - Vertical reaction for hard-tie rigging;
  - Vertical reaction for the drill pipe during downhole testing and sampling;
  - Vertical control of the drill pipe during drilling, downhole testing and sampling (hybrid seafloor/ vessel drilling);
- Heave compensator: apparatus to compensate (passive system) the drill pipe or logging tools for vertical motion of a vessel;
- Hard-tie rigging: special rigging system incorporating a seafloor template and a passive heave compensator, for heave-compensated drilling with low drill bit load and/or increased depth control of the drill bit.

Sampling Apparatus

Drive Sampling

- Drive-weight assembly: device consisting of hammer, hammer fall guide, anvil and hammer drop system;
- Hammer: impact mass, which is successively lifted and dropped to provide the energy that accomplishes sampler penetration;
- Hammer fall guide: guide arrangement for the fall of the hammer;
- Anvil: drive-head which the hammer strikes and through which the hammer energy passes into the sampling rods;
- Hammer drop system: pick-up and release mechanism by which lifting and dropping of the hammer is accomplished;
- Cathead: rotating drum in a rope-cathead hammer drop system around which a rope is wrapped to lift and drop the hammer by successively tightening and loosening the rope turns around the drum;
- Self-tripping release: hammer drop system that ensures a free fall of the hammer after lifting by a cable or rope;
- Free-fall winch: hammer drop system that permits a free release of the rotating drum of the winch around which a cable is wrapped to lift and drop the hammer;
- Hydraulic Percussion: hammer drop system that provides rapid impact hammer blows by fluid flow;
- Sampling rods: rods that connect the drive-weight assembly to the sampler head.

Push Sampling Apparatus

- Sampler insertion equipment: apparatus providing relatively rapid continuous penetration force;
- Reaction equipment: reaction for the sampler insertion equipment;
- Sampling rods: rods that connect the sampler insertion equipment to the sampler head.

Sampler

- Open-tube sampler: sampler with tube that is open at one end and fitted to the sampler head at the other end;
- Piston sampler: sampler with close-fitting sliding piston that is held stationary during penetration of a sample tube into the ground;
- Rotary core sampler:
  - Single tube core barrel: hollow steel tube with a head at the upper end threaded for drill pipe, and a threaded connection for the core bit at the lower end;
  - Double tube core barrel: assembly of two concentric steel tubes joined at the upper end by means of a swivel arranged to permit rotation of the outer tube without causing rotation of the inner tube; the upper end of the outer tube is threaded for drill pipe and the lower end is threaded for the core bit;
  - Double tube core barrel with retrievable inner tube: double tube core barrel that permits retrieval of the core-laden inner tube assembly to the surface through matching drill pipe without the need for withdrawal of the drill pipe;
- Sampler head: coupling between sampling rods and sample tube, and containing a non-return valve to allow free exit of water and air above sample;
- Sample tube: cylindrical tube with cutting edge or cylindrical tube fitted with separate cutting shoe;
- Liner: cylindrical plastic tube inserted in a sampler to preserve sample and reduce friction;
- Thin-walled sample tube: sample tube with area ratio of less than 15 % and inside clearance ratio of less than 1 %;
- Thick-walled sample tube: sample tube not meeting the requirements of a thin-walled sample tube;
- Core catcher: device that assists retention of the sample in the sampler;
- Cutting shoe: replaceable tapered cutting edge of sample tube to allow use of core catcher and the selection of inside clearance ratio and/or area ratio;
- Stroke length: maximum achievable sample length.

Table 1 shows dimensions of common samplers.

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Table 1: Dimensions of Samplers

Sampler Type	Inside Diameter D_1 [mm]	Outside Diameter D_2 [mm]	Inside Diameter D_3 [mm]	Wall Thickness [mm]	Area Ratio A_r [%]	Inside Clearance Ratio C_r [%]	Stroke Length[1] [mm]
Thin-walled 3 inch tube	72.0	76.0	72.0	2.0	10.2	0.0	950
Thin-walled 5° – 10° tube[2]	72.0	76.0	72.0	2.0	10.2	0.0	950
Thick-walled 3 inch tube	72.0	80.0	72.0	4.0	19.0	0.0	950
Thin-walled 2 inch tube	54.0	57.0	54.0	1.5	10.2	0.0	950 and 570
Thick-walled 2 inch tube	53.0	60.3	53.1	3.6	22.7	0.2	570
Piston Sampler[3]	72.0	76.0	72.0	2.0	10.2	0.0	845
3 metre EP Piston Sampler[3]	66.0	80.0	67.0	2.6	31.3	1.5	3000
Hammer Sampler 2 inch splitspoon	40.0	51.0	41.0	5.0	38.5	2.5	600
Rotary Core Sampler[4]	62.5	85.0	63.6	4.1	44.0	1.8	3000

Notes
1. "Length" dimension considers manufactured length. Re-use of a sampler may lead to shortening, for example to reshape the cutting edge.
2. Thin walled 5° - 10° tube is equivalent to conventional thin-walled 3 inch tube with nominal end taper, except for specially machined cutting edge with 5° and 10° taper to reduce sampling disturbance.
3. Tool can be assembled with liners and a range of cutting shoes; D_1, D_2, D_3, A_r and C_r can be varied.
4. Fugro Marine Core Barrel.

The definitions of area ratio and inside clearance ratio are as follows:

Area ratio: indication of volume of ground displaced by the sample tube, calculated as follows:

$$A_r = [(D_2^2 - D_1^2)/D_2^2] \cdot 100$$

where:

A_r = area ratio expressed as percentage
D_2 = outside diameter of the sample tube and/or cutting shoe
D_1 = inside diameter of the cutting shoe.

Inside clearance ratio: indication of clearance of sample inside the sample tube, calculated as follows:

$$C_r = [(D_3 - D_1)/D_1] \cdot 100$$

where:

C_r = inside clearance ratio expressed as percentage
D_3 = inside diameter of the flush portion of the sample tube or liner
D_1 = inside diameter of the cutting shoe.

The worst case of manufacturing tolerances applies for calculation of C_r.

Procedure

Figure 2 summarises the procedure for boreholes.

The procedure includes several stages, as follows.

Borehole Set-Up Stage

- Assignment of borehole details such as location, target borehole depth, types of apparatus, sequence of sampling;
- Positioning of drill rig at assigned location;
- Selection of drilling, logging, sampling or in situ testing stage.

The subsequent stage is one of the following.

Drilling

- Open hole or core drilling;
- Borehole logging, such as drill bits and drill fluids used, borehole size and depth, drilling observations, recording of drilling parameters – automated logging;
- Borehole water level, where practicable;
- Selection of subsequent drilling, sampling or in situ testing stage.

Borehole Geophysical Logging

- Tool deployment in-pipe or open hole;
- Logging of depth and geophysical parameter values;
- Selection of subsequent drilling stage.

In Situ Testing

- In situ test;
- Logging, such as test depth and test parameter values;
- Selection of subsequent drilling stage.

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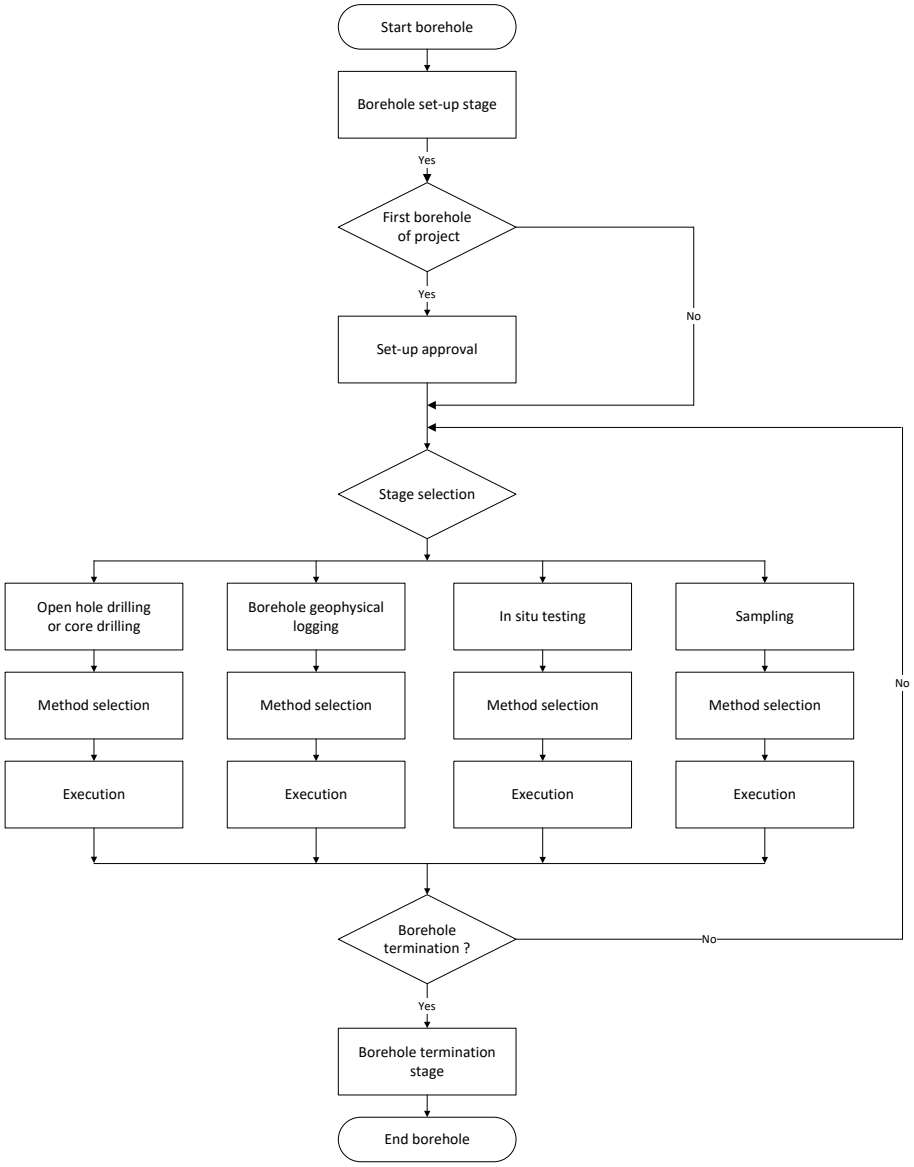

Figure 2: Flow chart

Sampling

- Sampling;
- Logging, such as sample depth and sample inspection at the time of sampling;
- Sample handling;
- Selection of subsequent drilling stage.

Borehole Termination Stage

- Termination of borehole;
- Backfilling of borehole, if appropriate;
- Data processing.

The set-up stage is at discretion of the equipment operator, particularly considering suitability of expected ground type(s), accessibility, risk of damage to equipment and safety of persons.

Set-up requires a reasonably flat, accessible, ground surface with a slope of 5° or less. Most onshore drilling systems have levelling facilities allowing a vertical start of drilling. Seabed reaction frames used for offshore drilling activities typically have no levelling facilities. Drill pipe passage through a seabed reaction frame must be (near-)vertical.

For over-water (marine/ offshore activities), additional accessibility considerations include:

- Minimum water depth for the selected pontoon, jack-up or vessel and the selected drilling, logging, testing and sampling equipment;
- Maximum water depth for the selected pontoon, jack-up or vessel;
- Maximum depth below water (sea) level for the selected drilling, logging, testing and sampling equipment;
- Metocean conditions, particularly wind, waves, currents.

The stages of logging, testing and sampling can include depth overlap. For example:

- Borehole geophysical logging can be conducted in a borehole that included earlier in situ testing and/or sampling;
- A sequence of (downhole) cone penetration tests can include short overlaps for acquiring seamless data coverage;
- A sample can be taken at a depth of a preceding CPT push (oversampling).

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The sampling stage may result in no-recovery or partial recovery of a sample due to unfavourable conditions for the deployment of a particular sampler. A subsequent sampling event at the same depth or immediately below the initial sampling depth is a separate sampling activity, unless specifically agreed otherwise or unless no specific evidence shows departure from the agreed procedure for the earlier activity.

Criteria for borehole termination are as follows, unless specifically agreed otherwise:

- As instructed by client;
- Reaching target penetration;
- Drilling progress rate of less than 1 m/hour based on half-hourly observation;
- Circumstances at discretion of system operator, such as risk of damage to apparatus or safety of persons;
- Whichever occurs first and as applicable.

Recording of Drilling Parameters

Recording of drilling parameters can be done manually or by automated logging. Common standards are ISO (2021) for onshore investigations and ISO (2014) for marine investigations. Standards typically consider functional checks of sensors for observation or recording of parameter values, i.e. no requirements for sensor calibration.

Recorded parameters typically include penetration rate, drill head rotational torque and drilling fluid pressure.

Records are qualitative. Recorded values depend on factors such as ground conditions, depth below ground surface, equipment deployment method, drilling method, type of drilling apparatus, characteristics of in-borehole drilling mud, drilling practices adopted by driller practice. In some cases, it can be possible to extract comparative information for interpretation of geotechnical conditions.

Results can (optionally) be graphically summarised in (geotechnical) logs, with depth below ground surface (seafloor) as vertical axis. The logs typically consider the actual drilling activities, i.e. exclude (time-based) data recorded during drilling interruptions.

ISO (2016) provides requirements for measuring-while-drilling for 'top-driven destructive drilling methods performed by a fully hydraulically powered drill rig', using a suite of calibrated sensors. The measuring-while-drilling method is rarely considered for marine investigations.

Sample Quality

Sampling techniques and samplers can be important for the evaluation of the results of laboratory tests.

Table 2 presents an assessment of typical sample class and Table 3 summarises the corresponding classification system for sample quality. The classification system recognises 5 classes based on feasibility of specific geotechnical identification and laboratory tests (CEN, 2007; ISO, 2014).

Comments on Class 1 and Class 2 fine-grained soil samples are as follows:

- Some sample disturbance is inevitable because of the required sampling process and subsequent on-site and laboratory sample handling;
- Silt soil is more sensitive to disturbance than clay soil;
- Sample disturbance typically increases with increasing total stress conditions applicable to the in situ soil. Negative pore pressures develop after sampling, upon reduction of total stresses. The resulting effective stresses within the sample cause sample disturbance. Sample disturbance may thus increase with sampling depth or with increasing water depth for offshore boreholes;
- Reduction in water pressure occurring after sampling causes a change in equilibrium between dissolved gasses, gas bubbles and gas hydrates, where present. The amount of gas release increases with water pressure. This may result in increased sample disturbance, in particular for deep-water sites.

Table 2: Sampling Technique and Sample Class

Sampler Type	Sample Class					
	Very Soft to Soft Clay	Stiff to Very Stiff Clay	Hard Boulder Clay	Loose to Medium Dense Silt/Sand	Dense Silt/ Sand	Weakly Cemented Soils and Rock
Thin-walled 3 inch tube	2 (1)	2 (1)	2 (1)	3 (2)	n/a (2)	n/a (2)
Thin-walled 5° – 10° tube	1	1	n/a	3 (2)	n/a (2)	n/a (2)
Thick-walled 3 inch tube	2	2	2	3 (2)	3 (2)	n/a (3)
Thin-walled 2 inch tube	2 (1)	2 (1)	2 (1)	3 (2)	2	n/a (3)
Thick-walled 2 inch tube	2	2	2	3 (2)	3 (2)	n/a (3)
Piston Sampler	1	1	n/a	3 (2)	n/a	n/a
3 metre EP Piston Sampler	2 (1)	2 (1)	2 (1)	3 (2)	n/a	n/a
Hammer Sampler 2 inch splitspoon	2	3 (2)	3 (2)	3	3	3
Rotary Core Sampler	n/a	3 (2)	3 (2)	n/a	n/a (3)	n/a (2)

Notes
1. Sampler type is according to Table 1 and sample class is according to Table 3.
2. Ground descriptions (e.g. stiff to very stiff clay) are according to ISO (2014) for applicability rating.
3. Sample class between brackets may be feasible under favourable conditions.
   n/a means not applicable or very low applicability.

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Table 3: Sample Application (Quality) Classes

Application (Quality) Class	Designation	Laboratory Test Feasibility
Class 1	Undisturbed	strength, stiffness and consolidation
Class 2	Undisturbed	layering, permeability, unit weight
Class 3	Disturbed	water content
Class 4	Disturbed	particle size analysis, Atterberg limits, soil type
Class 5	Disturbed	stratigraphy
Note The higher class includes the laboratory tests of the lower classes.		

In principle, sampling is continuous from bottom of borehole to maximum penetration. However, recovery ratios (ratio of recovered sample length to penetration) can differ from unity. Reasons include:

- Loss of ground from the bottom end of the sampler during retrieval;
- Compaction of ground due to vibration (hammer sampler), and/or handling;
- Settling of soil due to an inside clearance ratio C_r of the sampler of greater than zero;
- Soil expansion due to swelling-soil minerals, gas and/or stress release;
- Loss and/or segregation of very soft soil due to fluidisation;
- Plugging type behaviour during penetration (missing sample).

Sample quality may change with time and storage conditions. The type of soil or rock will influence the degree of change. For example, exposure to air may initiate chemical processes, such as rapid oxidisation of organic soil.

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Geotechnical Log

Introduction

A geotechnical log presents a one-dimensional, typically vertical, profile of ground strata and water level measurements, where applicable. In addition, it may include the principal details of operational activities for acquisition of the information shown on the geotechnical log.

Other terms for geotechnical log used in practice include core log, borehole log, drilling log, sample log, geohazard core log, geological log and wireline log.

Procedure

Preparation of geotechnical logs is based on ISO, CEN, BSI and ASTM standards.

The format and detail of a geotechnical log depends on an agreed project specification. Integration Level 1 of Table 1 applies, unless indicated otherwise.

A geotechnical log typically considers a specific purpose, for example presentation of geotechnical information for design of a pile foundation. The geotechnical log should not be used for another purpose without appropriate verification.

Table 1: Levels of Integration

Integration Level	Integration Type	Description
1	Bundled Information	Each data acquisition activity is interpreted and reported separately. No specific effort is made to consider and reconcile potential conflicts between information sources.
2	Stratigraphic Integration	This level of integration specifically focusses on achieving stratigraphic alignment between (1) sub-surface/sub-bottom profiles obtained by non-intrusive geophysical techniques (e.g. seismostratigraphy) and (2) stratigraphic interpretation from results of ground investigation obtained at specific locations (e.g. geotechnical soil unitisation). The stratigraphic alignment considers vertical zonation of a site.
3	Geotechnical Zonation	This level of integration provides a vertical and horizontal geotechnical zoning of a site. The horizontal zonation comprises a delineation and mapping of ´soil provinces´. Each soil province has a representative vertical soil profile and envelopes of ground characterisation such as shear strength, relative density, friction angle, unit weight, etc. The ´horizontal and vertical zoning´ facilitates selection of engineering criteria (e.g. geotechnical parameter values/ ranges) for analysis of trenchability, anchor holding capacity, foundation bearing resistance, etc.
4	Geotechnical Zonation and Analysis	This level of integration not only provides geotechnical zonation but also incorporates engineering assessments of specific project requirements such as bearing resistance, trenching resistance, anchor holding capacity, upheaval buckling resistance, scour potential, etc. These requirements are usually specific to the type of facility, construction method and project phase.

A basic geotechnical log can consist of descriptions limited to e.g. "soil" and "rock" or a value such as soil behaviour type index I_c, in combination with corresponding depths below ground surface or seafloor.

A comprehensive geotechnical log is an interpretation of selected, processed data. The procedure for interpretation typically includes ranking of information for quality and importance and, where applicable, selection of primary depth values and aligning other depth data with the primary reference. The selected data can include:

- geological information;
- 2D/ 3D geophysical data;
- results of nearby geotechnical investigation locations;
- borehole geophysical logging data;
- in situ test data;
- laboratory test results;
- drilling parameters such as torque, feed, drill fluid pressure and drilling time.

A geotechnical log can apply to:

- a single investigation location, such as a borehole or cone penetration test (CPT);
- a location cluster comprising two or more investigation locations;
- a soil province, site or region.

A comprehensive geotechnical log can include mm-scale geological descriptions, geotechnical strata, data points of laboratory test results and multiple profiles of borehole geophysical logging and interpretive results of in situ testing.

The level of detail and accuracy of a geotechnical log depends on factors such as sample size, quality, coverage of samples and test data and integration with any supplementary information. For example, interfaces between strata may be more gradual than a geotechnical log indicates. The selected method for data presentation can also influence the level of detail. For example, graphical presentation will be constrained by the selected vertical depth scale and horizontal scale(s). Any graphical presentation of test results considers values within the scale limits only. No automatic scaling applies, unless indicated otherwise. Tabular presentation of a geotechnical log (no linear depth scale and no fixed horizontal scales) imposes fewer constraints.

Example Information - Geotechnical Logs

Depth

A geotechnical log typically presents depths below ground surface or seafloor as positive values in the downward direction. Information can also be presented relative to a vertical datum such as mean sea level. This gives increasing values for elevation in the upward direction.

The penetration depth shown on a (vertical) geotechnical log is defined as the deepest point reached by drilling, sampling or in situ testing. The recovery depth is the deepest point for which investigation data (logging, sample and test data) are presented.

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In some cases, geotechnical logs can include geotechnical interpretations derived from geophysical data.

Unless indicated otherwise, recovery of a borehole tube sample or a core sample is assumed and shown to be continuous from the starting depth of sampling. Similarly, sample recovery for a seafloor sampler is assumed to be continuous from seafloor to recovery depth. In other words, the geotechnical log ignores possible plugging, flow-in and/or wash-out.

Geotechnical logs for a soil province, site or region can include multiple top and bottom depths for a single stratum or ground unit.

Operational Activities

A geotechnical log can include documentation of operational activities, such as details on drilling, sampling and in situ testing. Figure 1 shows examples for presentation of operational information.

Figure 1: Symbols for identification of samples and in situ tests

Drilling Parameters

Recording of drilling parameters for rotary drilling or percussion drilling can help characterisation of ground conditions such as cemented strata, weak rock and formations with cavities. Recording can be manually or by means of an automated recording system. Recorded parameter values are typically qualitative, i.e. no calibration of sensors would apply. Presentation of factual and/or interpreted results is usually in graphical format.

Rock Coring Parameters

ASTM International (2017) provides descriptions for rock core quality as follows:

TCR Total Core Recovery: the total core length divided by the core run length

SCR Solid Core Recovery: the total length of the pieces of solid core that have a complete circumference divided by the core run length

RQD Rock Quality Designation: the total length of the pieces of sound core over 100 mm long along the centreline divided by the core run lengths per stratum or core run; sound core includes core with obvious drilling breaks

I_F Fracture Index: spacing of natural discontinuities.

Table 2 shows a classification of rock quality according to ASTM International (2017).

Table 2: Classification of Rock Quality

RQD	Classification of Rock Quality
0 % to 25 %	Very poor
25 % to 50 %	Poor
50 % to 75 %	Fair
75 % to 90 %	Good
90 % to 100 %	Excellent

Geotechnical Description

Geotechnical description can be presented by text, numerical test values and by graphic logs.

Cone penetration test data allow software algorithms for geotechnical description of soil. Widely used systems are those by proposed by Robertson (2009, 2010, 2016). These systems include numerical test values, such as soil behaviour type index I_c as geotechnical description.

A geotechnical log can consist of or include a graphic log of ground conditions. Figures 2 through 4 present examples of symbols used in graphic logs.

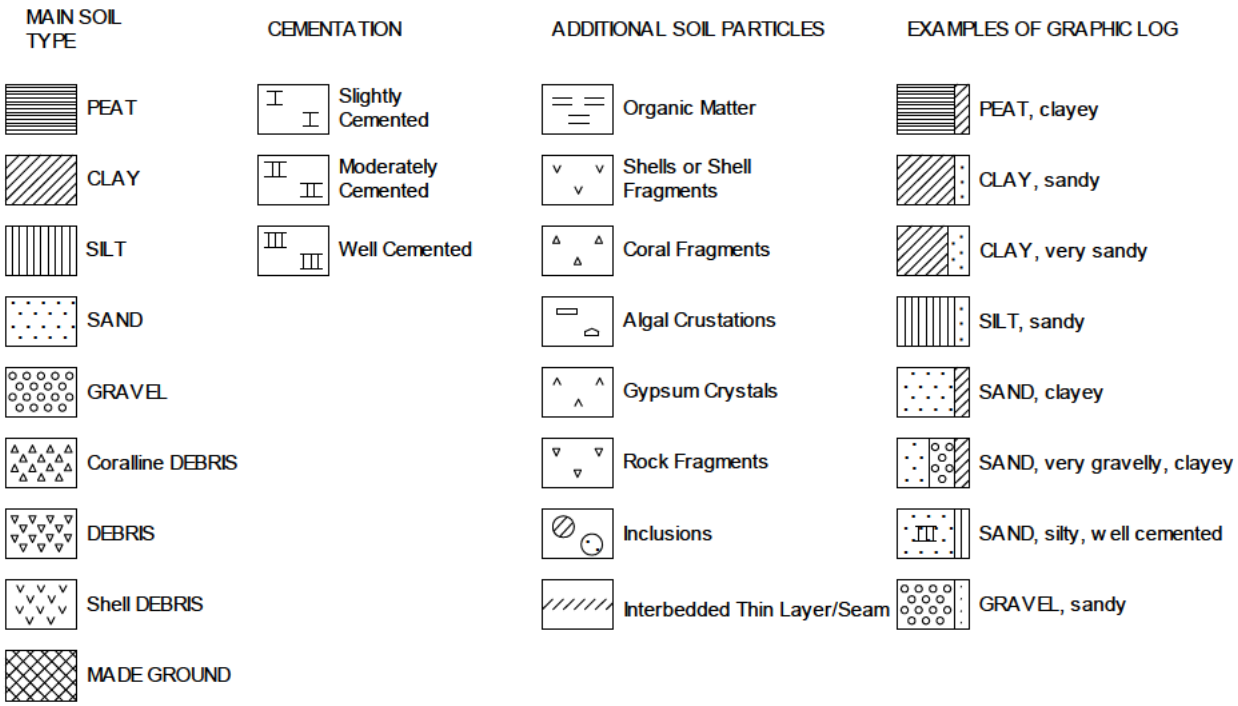

Figure 2: Symbols for soils

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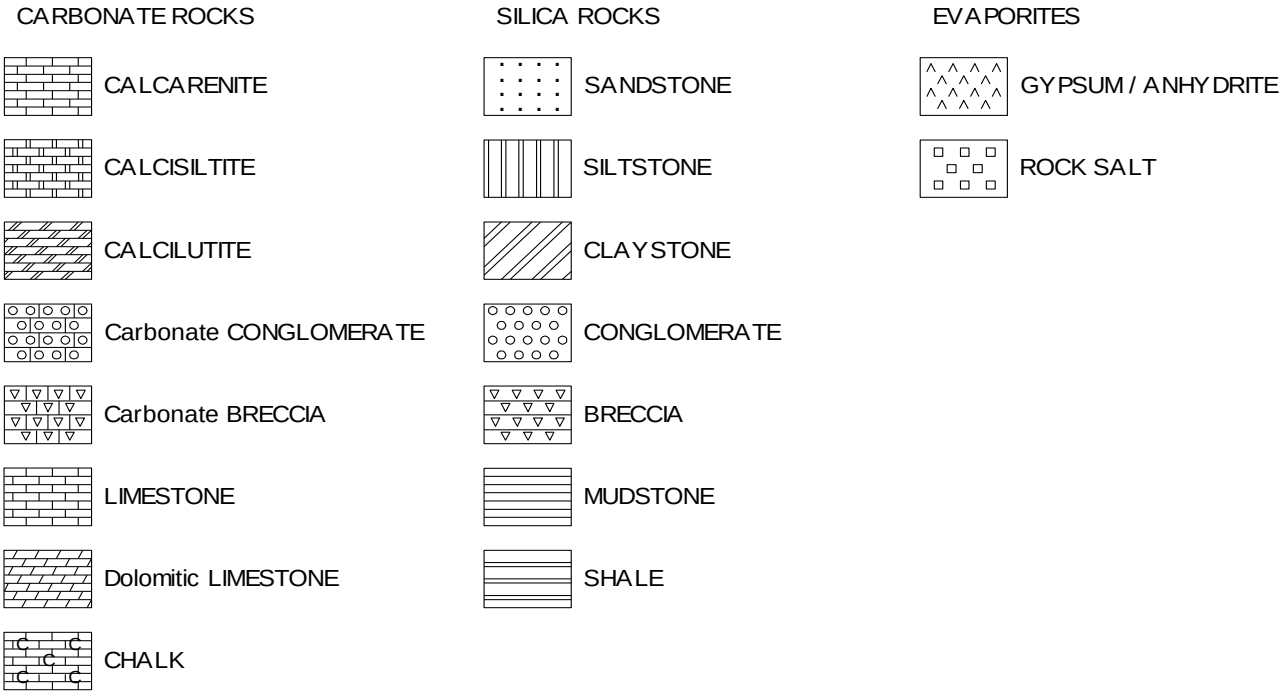

Figure 3: Symbols for sedimentary rocks

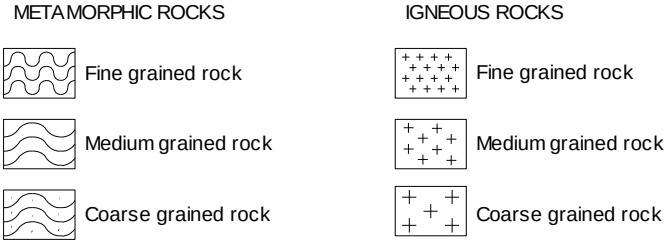

Figure 4: Symbols for metamorphic and igneous rocks

Water Level

Water level measurements taken in boreholes can be valuable. Interpretation of water levels requires due caution. They may or may not be representative of the ground water levels. In any case, water levels apply to the time and date of the measurements only. They will vary due to seasonal and other environmental influences, including construction activities.

Results of Geotechnical Tests and Correlations

A geotechnical log can include results of geotechnical tests and correlations, presented as numerical values and/or by graphic logs. Common examples are derived values of water content, soil unit weight, undrained shear strength, relative density and uni-axial compressive strength.

Test results and correlations for undrained shear strength are commonly shown for a main soil type shown as CLAY. Test results and correlations for relative density are commonly shown for a main soil type shown as SAND. In some cases, e.g. for transitional soils and layered soils, values of both undrained strength and relative density can be shown for CLAY, for SILT and for SAND. Specific decisions on presentation can be made by judgement and by algorithms. An example of a decision algorithm is CPT-based calculation using soil behaviour type index I_c.

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Cone Penetration Test

Introduction

The cone penetration test (CPT) involves the measurement of the resistance of ground to steady and continuous penetration of a cone penetrometer equipped with internal sensors. The measurements comprise penetration depth, cone resistance, sleeve friction and, optionally, pore pressure and inclination from vertical. These measurements permit interpretation of ground conditions.

CPT apparatus and procedures adopted by Fugro are in general accordance ISSMGE (1999), ASTM D5778-20, ISO 22476-1:2022, and ISO 19901-8:2014. General agreement also applies to Eurocode 7 (CEN, 2007).

Fugro offers CPT systems operated from (1) ground surface and seafloor (non-drilling deployment mode) and (2) downhole in a borehole (drilling deployment mode).

CPT Apparatus

General

CPT apparatus includes various parts as described below:

- Thrust machine: apparatus providing thrust to the push rods so that the recommended rate of penetration (20 mm/s) is controlled;
- Reaction equipment: reaction for the thrust machine;
- Push rod: thick-walled cylindrical tube used for advancing the penetrometer to the required test depth. Push rods may also consist of drill pipe;
- Friction-cone penetrometer (CPT): cylindrical terminal body mounted on the lower end of the push rods, including a cone, a friction sleeve and internal sensing devices for the measurement of cone resistance, sleeve friction and, optionally, inclination;
- Piezocone penetrometer (CPTU or PCPT): cylindrical terminal body mounted on the lower end of the push rods, including a cone, a friction sleeve, a filter and internal sensing devices for the measurement of cone resistance, sleeve friction, pressure and, optionally, inclination and temperature;
- Measuring system: apparatus and software, including sensors, data transmission apparatus, recording apparatus and data processing apparatus.

Deployment from Ground Surface or Seafloor

Specific additional apparatus for CPT deployment from ground surface and seafloor (non-drilling deployment) can include:

- Push rod casing: guide for the part of the push rods protruding above the soil, and for the push rod length exposed in water or soil, to prevent buckling when the required penetration pressure increases beyond the safe limit for the exposed upstanding length of push rods;
- Friction reducer to reduce soil friction acting on the push rods: (1) ring or special projections fixed on the outside of the push rods, with an outside diameter larger than the base of the cone and/or (2) injection of low-friction fluid from the push rod at a fixed distance above the cone penetrometer.

Downhole Borehole Deployment

Downhole CPT systems latch into a bottom hole assembly at the lower end of a drill pipe. System options are:

1. Operation of a downhole thrust machine by applying mud pressure in the borehole;
2. Remote control of a downhole thrust machine by hydraulic pressure transmitted through an umbilical cable connected to a surface-based pump unit, together with;
3. Application of thrust to drill rods where CPT apparatus and a short push rod are latched in the bottom hole assembly; the thrust machine is at ground surface or seafloor.

Downhole CPTs require drilling apparatus for advancing the borehole. The maximum CPT stroke is generally 1.5 m or 3 m.

Data recording can be surface-based and/or downhole.

Cone Penetrometer

Typical features of Fugro penetrometers (Figure 1) include:

- Cone base areas of 500 mm^2, 1000 mm^2 or 1500 mm^2;
- Cone and friction sleeve sensors placed in series, i.e. subtraction-type penetrometers;
- Pore pressure measurements either at the face of the cone (u_1 location) or at the cylindrical extension of the cone (u_2 location). Multiple-sensor penetrometers (u_1, u_2 and u_3 locations) are also available. The u_3 location is immediately above the friction sleeve;
- Inclinometer;
- Temperature sensor, e.g. for cone penetrometer class 0 specified in ISO 22476-1:2022;
- Storage of signals from the penetrometer in digital form for subsequent computer-based processing and presentation.

Apparatus for Additional Measurements

Add-on apparatus (and procedures) can apply to specific additional measurements, refer to section 'Additional Measurements' below.

Procedure

General

Figure 2 summarises the test procedure. The procedure includes several stages. The stage of Additional Measurements is optional.

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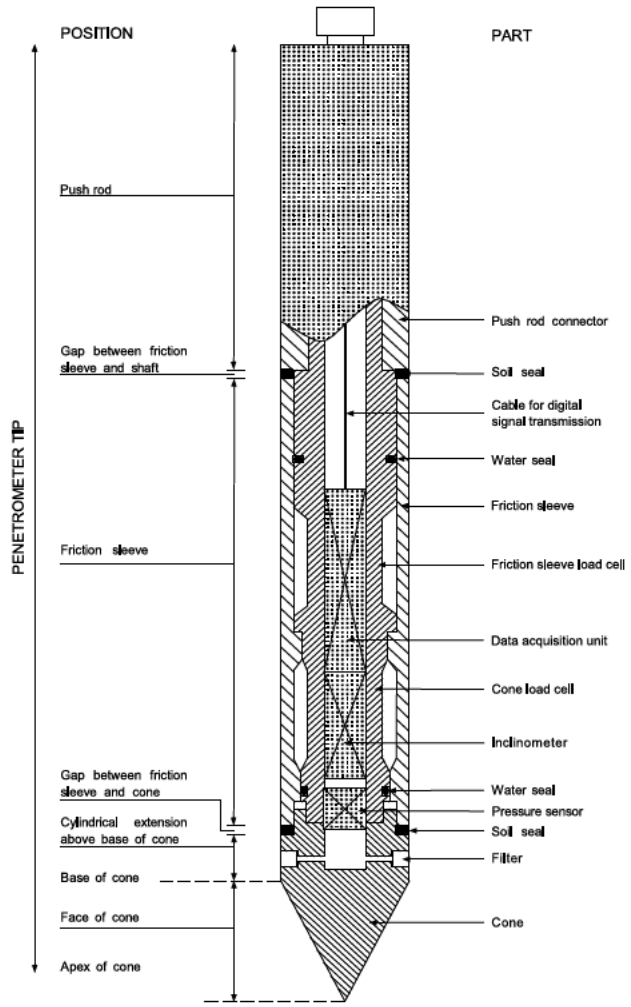

Figure 1: Piezocone penetrometer

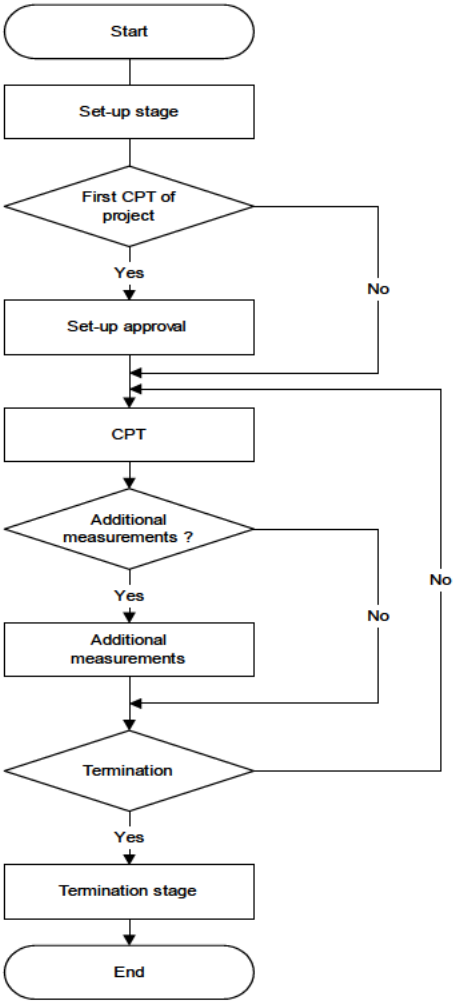

Figure 2: Flow chart

Set-up Stage and Termination Stage

The set-up stage is at discretion of the equipment operator, particularly considering suitability of expected ground type(s), accessibility, risk of damage to equipment and safety of persons.

Set-up requires a reasonably flat, accessible, ground surface with a slope of 5° or less. Most onshore thrust machines have levelling facilities allowing a vertical start of penetration. Seabed frames used for offshore CPT activities have no levelling facilities, i.e. start of penetration may not be vertical.

For over-water (marine/ offshore activities), additional accessibility considerations include:

- Minimum water depth for the selected pontoon, jack-up or vessel and the selected test equipment;
- Maximum water depth for the selected pontoon, jack-up or vessel;
- Maximum depth below water (sea) level of selected test equipment;
- Metocean conditions, particularly wind, waves, currents.

The set-up stage typically includes selection of equipment and procedures according to a required type of cone penetrometer, application class, cone penetrometer class, test category and data processing/ submission.

The set-up stage or the termination stage includes the location survey, i.e. the determination of the coordinates and the ground surface elevation (or the water depth).

The set-up stage and the termination stage for a downhole CPT include lowering of the CPT apparatus into the borehole and lifting respectively. Most projects require multiple downhole tests in a single borehole.

For piezocone testing, the set-up stage also includes the following steps:

- Office-based or site-based: de-airing of the filter in glycerine by application of 24-hour vacuum and storage in a glycerine-filled container;
- On-site: glycerine filling of hollow space in the cone penetrometer and subsequent mounting of the filter;
- On-site: application of a flexible membrane around the filter to prevent loss of saturating fluid prior to the start of a test.

Land-based tests may include specific measures to help retention of filter saturation during penetration of partially saturated zones. Relaxation of requirements typically applies to offshore tests where water pressures will force entrapped air into solution.

Criteria for test termination are as follows, unless specifically agreed otherwise:

- As instructed by client;
- Reaching target penetration;
- Reaching maximum capacity of the thrust machine, reaction equipment, push rods and/or measuring sensors;

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- Sudden increase in penetrometer inclination;
- Risk of damage to apparatus or safety of persons, at discretion of equipment operator or as determined by software algorithms;

whichever occurs first and as applicable. Note that ASTM and ISO standards provide no specific requirements for maximum penetrometer inclination from vertical. A value of 15° is commonly considered.

Application Classes – ISO 19901-8:2014

Table 1 summarises application classes specified in ISO 19901-8:2014 for offshore and nearshore CPTs. The allowable minimum accuracy of a measured parameter is the larger value of the two quoted. A percentage value applies to the measured value and not to the measuring range.

Table 1: Application classes (ISO 19901-8:2014)

Application Class	Parameter	Allowable Minimum Accuracy
1	Cone resistance	35 kPa or 5 %
	Sleeve friction	5 kPa or 10 %
	Pore pressure	25 kPa or 5 %
2	Cone resistance	100 kPa or 5 %
	Sleeve friction	15 kPa or 15 %
	Pore pressure	50 kPa or 5 %
3	Cone Resistance	200 kPa or 5 %
	Sleeve friction	25 kPa or 15 %
	Pore pressure	100 kPa or 5 %

The concept of application classes considers intended soil conditions for selection of an application class. For example, Application Class 1 of ISO 19901-8:2014 can be selected for 'very soft to soft soil deposits', which is approximately equivalent to $q_c < 0.5$ to $q_c < 1$ MPa. In other words, Application Class 1 should not apply to 'mixed bedded soil profiles with weak to strong layers'.

The accuracy values apply to seafloor as reference. They are uncoupled from uncertainty of spatial position below ground surface or seafloor.

Historically, the concept of application classes was based on an international reference test procedure (ISSMGE, 1999), which specifies 'performance' criteria for cone penetration test measurements. The test results should meet the requirements of one of the application classes.

The following comments apply:

- Accuracy is the 'closeness of a measurement to the true value of the quantity being measured'. It is the accuracy as a whole that is ultimately important not the individual parts. Precision is the 'closeness of each set of measurements to each other'. The resolution of a measuring system is the 'minimum size of the change in the value of a quantity that it can detect'. It will influence the accuracy and precision of a measurement.
- Application Class 3 typically represents industry practice. They are approximately equivalent to the more implicit requirements of ASTM International. Class 3 applies, unless specifically agreed otherwise.

Differences in interpretation about compliance with the ISO box values for accuracy became apparent after publication of a predecessor of ISO 22476-1:2022 and, subsequently, publication of ISO 19901-8:2014. Unfortunately, the interpretational challenges emerged from contractual disputes, unnecessary re-work and CPT results assigned higher confidence than actual (e.g. Peuchen and Parasie, 2019).

The zero drift of a measured parameter can be compared with the allowable minimum accuracy according to the selected application class, per test. This comparison considers the maximum range of values of q_c, f_s and, where applicable, u_1 or u_2 for calculation of the percentage box values (Table 1: Application classes (ISO 19901-8:2014)). Zero drift' of a measured parameter is an approximate performance indicator for the measuring system (Peuchen and Terwindt, 2014). Zero drift is the absolute difference of the zero readings, reference readings or zero reference reading of a measuring system between the start and completion of the cone penetration test. The reference readings can be taken at (1) atmospheric pressure at ground surface or above water level or (2) under hydrostatic water pressure close to seafloor.

ISO 19901-8:2014 is under revision (2023). The revision will probably lead to replacement of the application classes by a prescriptive approach similar to that specified in ISO 22476-1:2022, i.e. method-based requirements and cone penetrometer classes.

Cone Penetrometer Classes and Test Categories – ISO 22476-1:2022

The applicability of ISO 22476-1:2022 is onshore and nearshore. The standard allows selection of cone penetrometer classes and test categories, i.e. method-based criteria. Compliance with a particular cone penetrometer class and test category then provides some indication for uncertainty of CPT results.

Cone penetrometer classes rely on results of detailed laboratory calibration and verification of cone penetrometers. The results determine compliance of a cone penetrometer with one of four cone penetrometer classes (Table 2). A cone penetrometer can conform to more than one cone penetrometer class, for the case of multiple intervals for calibration.

Input criteria for the cone penetrometer classes include:

- Minimum measurands per cone penetrometer class (Table 2);
- Laboratory cone resistance and sleeve friction: (1) selected uncertainty components for axial force, (2) resolution and output stability, (3) verification values of ambient temperature stability, transient temperature stability and bending influence;
- Laboratory pore pressure: (1) selected uncertainty components for water (or gas) pressure, (2) resolution and output stability, (3) verification values of ambient temperature stability, transient temperature stability and bending influence;
- Inclination: expanded measurement uncertainty for inclination values determined in a calibration laboratory.

Table 2: Required measurands per cone penetrometer class (ISO 22476-1:2022)

Cone Penetrometer Class	q_c	f_s	u_2	T
0	√	√	√	√
1	√	√	√	
2	√	√		
3	√	√		

Notes
q_c = cone resistance
f_s = sleeve friction
u_2 = pore pressure (and/or u_1)
T = temperature

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Test categories consider requirements for (1) cone penetrometer class and (2) reference readings and output stability of a cone penetrometer recorded just before the cone penetrometer penetrates the ground and just after the cone penetrometer leaves the ground (Table 3). The requirements for pore pressure u_2 (or u_1) apply according to cone penetrometer class (Table 2).

Table 3: Requirements for test categories (ISO 22476-1:2022)

Test Category	Reference Readings [kPa]	Output Stability [kPa]	Cone Penetrometer Class
A	$\Delta q_{c;0} \leq 15$	$2\hat{u}_{qc} \leq 1$	0
	$\Delta f_{s;0} \leq 5$	$2\hat{u}_{fs} \leq 0.5$	
	$\Delta u_{2;0} \leq 3$	$2\hat{u}_{u2} \leq 0.5$	
B	$\Delta q_{c;0} \leq 35$	$2\hat{u}_{qc} \leq 5$	0 or 1
	$\Delta f_{s;0} \leq 5$	$2\hat{u}_{fs} \leq 1.5$	
	$\Delta u_{2;0} \leq 10$	$2\hat{u}_{u2} \leq 3$	
C	$\Delta q_{c;0} \leq 100$	$2\hat{u}_{qc} \leq 11$	0, 1, or 2
	$\Delta f_{s;0} \leq 15$	$2\hat{u}_{fs} \leq 3$	
	$\Delta u_{2;0} \leq 25$	$2\hat{u}_{u2} \leq 8$	
D	$\Delta q_{c;0} \leq 200$	$2\hat{u}_{qc} \leq 33$	0, 1, 2, or 3
	$\Delta f_{s;0} \leq 25$	$2\hat{u}_{fs} \leq 5$	
	$\Delta u_{2;0} \leq 50$	$2\hat{u}_{u2} \leq 16$	

Notes
$\Delta q_{c;0}$ = difference in reference readings for cone resistance q_c
$\Delta f_{s;0}$ = difference in reference readings for sleeve friction f_s
$\Delta u_{2;0}$ = difference in reference readings for pore pressure u_2 (or u_1)
$2\hat{u}_{qc}$ = output stability for q_c
$2\hat{u}_{fs}$ = output stability for f_s
$2\hat{u}_{u2}$ = output stability for u_2 (or u_1)

The difference in reference readings (e.g. $\Delta q_{c;0}$) of a sensor is calculated from sensor output recorded at a frequency of ≥1 Hz, as follows:

- Subtracting the mean value of reference readings of a particular sensor (e.g. sensor for q_c), for a period of one minute shortly before the penetration phase from the mean value of reference readings for a period of one minute shortly after the extraction phase, expressed as an absolute value;
- Cone penetrometer is vertical and under no load, atmospheric or selected ambient water pressure;
- Cone penetrometer is under temperature conditions close to ground temperature.

Calculation of output stability (peak-to-peak) of a sensor (e.g. $2\hat{u}_{qc}$) makes use of reference readings as described above. The calculation considers the larger value of subtracting the maximum and minimum sensor values for a period of one minute shortly before the penetration phase and for a period of one minute shortly after the extraction phase.

Results

CPT Parameters

Presentation of results from cone penetration tests typically includes:

- CPT parameters q_c, f_s and R_f versus depth below ground surface or versus elevation;
- Additional CPTU parameters u_1 or u_2 and, optionally, q_t, q_n, B_q, Q_t, Q_{tn}, F_r, I_c, I_B and CD for tests with pore pressure measurements;
- Inclination i for tests with inclination measurements;
- Standard graphical format and digital (tabular) ASCII or AGS formats.

Presentation of temperature T versus depth only applies to test results meeting the requirements of both cone penetrometer class 0 of Table 2 and test category A of Table 4.

Most standards specify scales for graphical presentation as follows:

- Axis for penetration depth z: 1 scale unit = 1 m;
- Axis for cone resistance q_c, corrected cone resistance q_t and net cone resistance q_n: 1 scale unit = 2 MPa or 0.5 MPa;
- Axis for sleeve friction f_s: 1 scale unit = 50 kPa;
- Axis for friction ratio R_f: 1 scale unit = 2 %;
- Axis for pore pressure u: 1 scale unit = 0.2 MPa or 0.02 MPa;
- Axis for pore pressure ratio B_q: 1 scale unit = 0.5.

Graphical presentation aims for these scale units and scale ratios, where suitable and practicable.

The reference level of a test is (1) the ground surface for onshore tests, (2) the seafloor for nearshore and offshore tests. Data processing presumes a hydrostatic pore pressure profile relative to seafloor, unless specifically indicated otherwise. The definition of CPT parameters is as follows:

z = penetration depth relative to ground surface or seafloor, corrected for inclination from vertical (i) where a test includes inclination measurements, as follows:

$$z = \int_0^l \cos i \cdot \mathrm{d}l$$

where:

- z = penetration depth for the conical base of the cone penetrometer
- l = recorded penetration length
- i = recorded inclination from vertical

q_c = cone resistance relative to the reference level of the test.

f_s = sleeve friction relative to the reference level of the test. A calculated depth correction applies so that the presented sleeve friction corresponds with the cone depth.

f_t = corrected sleeve friction relative to the reference level of the test. Sleeve friction is corrected for pore pressures acting on the end areas of the friction sleeve

$$f_t = f_s - \frac{(u_2 * A_{sb} - u_s * A_{st})}{A_s}$$

or simplified to:

$$f_t = f_s - u_2 \frac{(A_{sb} - A_{st})}{A_s} \quad \text{or}$$

$$f_t = f_s - (u_2 * a_{fs})$$

where:

- A_{sb} = cross sectional area in the gap between the friction sleeve and the cone
- A_{st} = cross sectional area in the gap above the friction sleeve
- A_s = surface area of the friction sleeve
- a_{fs} = net area ratio of the friction sleeve $(A_{sb} - A_{st})/A_s$

R_f = ratio of sleeve friction to cone resistance (f_s/ q_c). This calculated ratio is for the cone depth.

R_{ft} = corrected friction ratio (f_s/ q_t). The ratio f_t/q_t applies if f_t is known.

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u_1 = pore pressure at the face of the cone, relative to the reference level of the test.

u_2 = pore pressure at the cylindrical extension above the base of the cone or in the gap between the friction sleeve and the cone, relative to the reference level of the test.

u_3 = pore pressure immediately above the friction sleeve or in the gap above the friction sleeve, relative to the reference level of the test.

Commonly, no measurement of u_3 applies. An estimate can be obtained using $u_3 = 0.7\ (u_2 - u_0) + u_0$ according to SGI (1991). This estimate is independent of positive or negative values of $u_2 - u_0$.

u_0 = hydrostatic pore pressure at the cone, relative to the phreatic surface or the seafloor. This is a calculated value.

q_t = corrected cone resistance (also called total cone resistance). This includes corrections for hydrostatic and transient pore pressures, and cone construction. The corrected cone resistance is relative to ground surface or seafloor:

$q_t = q_c + (1\text{-}a)u_2$ or

$q_t = q_c + (1\text{-}a)[K(u_1 - u_0) + u_0]$

Historically, equations for downhole tests were:

$q_t = q_c + (1\text{-}a)u_2 + u_{0i}$ or

$q_t = q_c + (1\text{-}a)[K(u_1 + u_{0i} - u_0) + u_0] + a * u_{0i}$

where:

a = net area ratio of the cross-sectional steel area at the gap between cone and friction sleeve to the cone base area. This ratio is penetrometer-type dependent. The a-factor indicates the effect of pore pressure on unequal cross-sectional areas of the cone.

u_{0i} = hydrostatic pore pressure at the bottom of the borehole, relative to seafloor. This is a calculated value.

K = adjustment factor for the ratio of pore pressure at the cylindrical extension above the base of the cone to pore pressure on the cone face $K = (u_2 - u_0)/(u_1 - u_0)$

The term $u_2 - u_0$ refers to excess pore pressure (with respect to hydrostatic pore pressure). Common symbols for excess pore pressure are du_2 and Δu_2. Similarly, du_1 and Δu_1 can represent the term $u_1 - u_0$.

The K-factor is only of interest for processing of CPTU results with pore pressure measurement at the cone face (u_1). The factor depends on soil characteristics such as fabric, overconsolidation ratio, compressibility and crushability. The K-factor (Peuchen et al., 2010) can be estimated from:

$$K = 0.91\mathrm{e}^{-0.09{Q_t}^{0.47}}\left(\frac{1}{1+F_r(0.17+0.061(Q_t-21.6)^{1/3})} - \mathrm{e}^{-2F_r}\right)$$

q_n = $q_t - \sigma_{vo}$ = net cone resistance. This includes corrections for hydrostatic and transient pore pressures, in situ stress, and cone construction. The symbol for q_n may also be q_{net}.

where:

σ_{vo} = total in situ vertical stress at the cone base, relative to ground surface or seafloor. This is a calculated value.

B_q = pore pressure ratio $B_q = (u_2 - u_0)/q_n$ or

$B_q = K(u_1 - u_0)/q_n$

Q_t = q_n/σ'_{v0} = normalized cone resistance

where:

σ'_{v0} = effective in situ vertical stress at the cone base, relative to ground surface or seafloor. This is a calculated value.

Q_{tn} = normalized cone resistance with variable stress exponent n, where:

$$Q_{tn} = [(q_t - \sigma_{v0})/P_a](P_a/\sigma'_{v0})^n$$

$$n = 0.381\ (I_c) + 0.05\ (\sigma'_{vo}\ /\ P_a) - 0.15 \quad \text{and} \quad n \le 1$$

(Zhang et al., 2002)

where:

P_a = atmospheric pressure

F_r = f_t/q_n = normalized friction ratio

U_2 = normalized excess pore pressure $(u_2 - u_0)/\sigma'_{v0}$

I_B = soil behaviour type index (Robertson, 2016)

$I_B = 100(Q_{tn} + 10)/(70 + Q_{tn}F_r)$

I_c = soil behaviour type index (Robertson and Wride,1998)

$I_c = [(3.47 - \log Q_{tn})^2 + (\log F_r + 1.22)^2]^{0.5}$

$I_{SBT} = [(3.47 - \log (q_c/P_a))^2 + (log\ R_f + 1.22)^2]^{0.5}$

I_{SBT} = soil behaviour type index (Robertson, 2010)

$I_{SBT} = [(3.47 - \log (q_c/P_a))^2 + (\log R_f + 1.22)^2]^{0.5}$

CD = contractive-dilative boundary (Robertson, 2016)

$CD = (Q_{tn} - 11)(1 + 0.06F_r)^{17}$

Presented values for u_2, q_t, q_n and B_q may be denoted by u_2^*, q_t^*, q_n^*, B_q^*, Q_t^* and F_r^* if u_2 is derived rather than measured, for example if derived by applying a K-factor.

Pore pressure u_2 at the cylindrical extension is commonly assumed equal to u_{2g} in the gap. The assumption $u_2 = u_{2g}$ is probably reasonable for deepwater CPTs and associated high values of ambient pressure that promote saturated conditions in the gap. A similar comment applies to u_3. Note that CPTU saturation procedures apply to the pore pressure measuring system only. These procedures exclude the gaps below and above the friction sleeve.

Some deployment systems allow monitoring of CPT parameters in reverse mode, i.e. upon retraction of the cone penetrometer. This optional feature presents additional information that can improve interpretation of ground behaviour, for example strength sensitivity of fine-grained soil.

Metrological Confirmation

CPT results include information on metrological confirmation. Examples covered by CPT standards include reporting of

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application class, cone penetrometer class, test category and reference readings.

The ISO standard on metrological confirmation (ISO 10012:2003) provides the general framework for assessment of performance compliance.

Cone penetration test standards can follow a 'prescriptive' approach, whereby specific detailed measures provided a 'deemed to comply' practice. ASTM D5778-20 and ISO 22476-1:2022 provide examples of this approach.

The level of detail required by standards can be high. For example, ISO 22476-1:2022 includes detailed procedures for calibration and verification of CPT systems, with normative references to ISO/IEC 17025:2017. Fugro's calibration laboratory holds formal accreditation for cone penetrometer calibration and verification according to ISO/IEC 17025:2017.

Peuchen and Terwindt (2014, 2015) provide guidance on uncertainty estimation for cone penetration test results. The calculation model for uncertainty estimates for q_c, f_s and u considers the following uncertainty contributions, where applicable: (1) force and pressure sensors, (2) geometry of the cone penetrometer, (3) effects from ambient and transient temperature, (4) non-axial force on cone penetrometer (bending moment), (5) ambient fluid pressure in soil and (6) zero offsets for q_c, f_s and u relative to seafloor.

Temperature Stability of Cone Penetrometer

Uncertainty considerations for strongly layered soils should allow for heat flux phenomena. Heat flux gives an apparent shift in cone resistance. For example, friction in dense sand causes a cone to heat by about 1°C/MPa cone resistance. Resulting heat flux changes cone resistance by an apparent shift in the order of 100 kPa to 200 kPa for a penetrating probe going from dense sand into clay. This is a temporary change lasting about 5 minutes. Penetration interruption can serve as mitigation measure for transient temperature effects. The incorporation of one or more add-on temperature sensors in a cone penetrometer, and associated data algorithms, can reduce the effects from ambient and transient temperature fluctuations (Peuchen et al. 2020).

Pore Pressures

A CPTU pore pressure measuring system is intended for use in water-saturated uncemented fine-grained soil. Pore pressure measurements (u) are commonly assumed to represent pore water pressures. This assumption is reasonable for soils saturated under in situ stress conditions and remaining saturated during penetration of the cone penetrometer.

Pore pressure results obtained for ground conditions such as partially saturated soils, very dense sands and cemented soils may not be representative and/or repeatable. For example, stiffness differences between the steel components of the cone penetrometer and the piezocone filter can affect results for very dense sands.

Loss of saturation of the pore pressure measuring system can occur during a test (Peuchen et al. 2020). Loss of saturation usually causes a sluggish pore pressure response during penetration of ground below the zone causing desaturation of the pore pressure measuring system. Reasons for loss of saturation include:

- Penetration of partially saturated ground, for example ground containing significant amounts of gas;
- Reduction of pore pressure to below in situ pore pressure, causing gas in solution to become free gas;
- Penetration interruption for a stationary in situ test or for add-on of a push rod, that will cause:
  - Abrupt cone penetrometer deceleration and acceleration, with a possibility of upward movement of the cone penetrometer
  - Change of stress conditions around the cone penetrometer, including pore pressure and gas migration where applicable
  - Small volume change of the gaps below and above the friction sleeve of the cone penetrometer
- For u_2 filter position: proximity of gap between cone tip and friction sleeve, i.e. net area ratio $a < 1$. This gap may not be water-saturated, which in turn can lead to (1) substantial, local (undesired) pore pressure gradients and (2) loss of saturation of the u_2 filter itself;
- Measurement of negative pore pressures such that cavitation occurs; for example, this is not uncommon for a piezocone filter located at the cylindrical extension above the base of the cone (u_2 location), at the time of penetration of dense sand or overconsolidated clay layers.

Re-saturation of a pore pressure measurement system can take place upon further penetration into soil. Particularly, re-saturation may take place in saturated low-permeability soils (clays) that are normally consolidated or lightly overconsolidated and where the gap can become saturated by adequate supply of water and/or water pressure.

Measured pore pressures affected by desaturation of the pore pressure measurement system may not be representative of soil behaviour. Consequently, derived parameter values that use pore pressure may also not be representative.

Shallow Penetration

Shallow penetration will affect CPT measurements. Values of q_c, f_s and u for initial penetration of a cone penetrometer below ground surface, seafloor or bottom of a borehole will differ from a fully embedded cone penetrometer. As a general guide, initial penetration effects can be expected for a distance of about 8 times the diameter of the cone penetrometer for q_c, u_1 and u_2, and for a distance of about 15 times the diameter of the cone penetrometer for f_s. Initial penetration effects can be deeper for downhole borehole deployment. This is because of (1) complex ground stress conditions immediately below the required borehole and (2) borehole-induced ground disturbance that cannot be avoided.

Use of reaction equipment will affect stress conditions for shallow penetration. Particularly, offshore conditions may include extremely soft ground at seafloor. Soil disturbance, pore pressure build-up and consolidation of near-surface soft soil may take place.

Penetration Rate

CPT standards typically provide limits of ± 5 mm/s for a nominal penetration rate of 20 mm/s. Considerations include:

- A typical thrust machine provides a push speed with an uncertainty within ± 5 mm/s under favourable conditions. Under adverse conditions, penetration rates may be outside these limits, for example with strongly varying thrust and towards the thrust limit of a thrust machine;
- The penetration rate is not necessarily equal to the push speed because of inevitable vertical movements of the thrust machine and length variation and bending of the push-rod string.

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Penetration Interruption

A penetration interruption may be unavoidable, for example to add a push rod or to perform a pore pressure dissipation test. This will affect test results.

Consolidation of low-permeability soil around a cone tip is of particular interest. A stationary cone penetrometer can apply local stresses that approach failure conditions, i.e. about 9 times the undrained shear strength or about 2 times the in situ mean effective stress. Pore pressure re-distribution and dissipation occur, resulting in a local increase in undrained shear strength and hence cone (bearing) resistance. A doubling of cone resistance may not be unreasonable for 100 % consolidation. Supplementary considerations include:

- Small downward movement of a penetrometer (order of millimetres) during a test can contribute to maintaining local stresses approaching failure conditions;
- Soil consolidation around a cone penetrometer may lead to soil/penetrometer adhesion that is sufficient to give an increase in 'cone' diameter. Resumption of penetration will lead to loss of adhered soil, usually within an equivalent distance of a few times the cone diameter;
- A low B_q value may imply partially drained penetration conditions. It is likely that any steady-state penetration conditions will not apply instantaneously upon resumption of penetration;
- Measuring sensors in a probe generate heat, but this is probably not significant for any stationary measurement. Fugro's strain-gauge load sensors are compensated for ambient temperature fluctuations.

Depth Measurement for Offshore Conditions

Table 4 presents depth accuracy classes according to ISO 19901-8:2014. The type of uncertainty is undefined (e.g. combined standard uncertainty or expanded measurement uncertainty with a coverage factor $k = 2$).

Peuchen and Wemmenhove (2020) present a probabilistic approach to depth uncertainty assessment for in situ testing data points, with reference to these accuracy classes.

Offshore definition of the seafloor (ground surface) is difficult for extremely soft ground at seafloor (ISO 19901-8:2014). Penetration of the reaction equipment into a near-fluid zone of the seabed may take place unnoticed. Such settlement affects the start of penetration depth z. Also, settlement may continue at the time of testing.

Downhole CPT systems rely on depth control applicable to borehole drilling. Depth control according to Z2 of Table 4 is typically feasible for drilling systems deployed from a fixed platform, for example a jack-up. This value excludes uncertainty associated with determination of seafloor level. Drilling control from floating equipment, for example a geotechnical survey vessel, may be subject to the additional influence of waves and tides. Z2 is typically feasible for favourable conditions. Z3 or Z4 may apply for adverse conditions.

Table 4: Depth accuracy classes (ISO 19901-8:2014)

Depth Accuracy Class	Maximum Data Point Depth Uncertainty [m]
Z1	0.1
Z2	0.5
Z3	1.0
Z4	2.0
Z5	> 2.0

Zero-Correction for Offshore Conditions

Water pressures generate significant values of cone resistance and pore pressure. The standardised practice is to correct these reference readings to zero at seafloor. CPT systems for non-drilling mode and for seafloor drilling mode allow zero-correction to hydrostatic conditions prior to the start of a test, typically with a zero-correction uncertainty approaching the resolution of the CPT system. Downhole borehole CPT systems latch into the lower end of a drill pipe. The pressure conditions in the drill pipe may not be in full equilibrium with the surrounding ground water pressure and zero-correction will be subject to increased uncertainty, i.e. uncertainty for pore pressure in the order of 100 kPa for deepwater tests (Peuchen, 2000). This uncertainty depends on factors such as the free-flow and viscosity of drill fluid between the drill bit and the seafloor. The uncertainty typically decreases with decreasing depth of the drill bit below sea level and below seafloor. Uncertainty for the zero-correction of cone resistance is approximately equivalent, but by a factor representing the net area ratio effect.

Deepwater Tests

A deepwater environment presents some favourable conditions for cone penetration tests, notably temperature. Ambient temperature conditions are practically constant and the measuring system has ample time to adjust to these temperatures. In addition, transient heat flow phenomena in a cone penetrometer are usually not applicable. This is because a cone penetrometer accumulates negligible (frictional) heat when penetrating the generally prevalent soils of very soft consistency.

Deepwater (piezocone) pore pressure measurements are essentially similar to shallow-water measurements, with the exception of an increased measuring range for pore pressure leading to some reduction in sensor accuracy. Saturation of a pore pressure measuring system is excellent for a deepwater environment, as the high pressures will force any gas bubbles into solution.

Currently available evidence indicates that a high-quality subtraction-type cone penetrometer is adequate for very soft soil characterisation to a water depth of 3000 metres and probably beyond.

Additional Measurements

Friction-cone and piezocone penetrometers allow specific additional measurements, such as friction set-up tests, pore pressure dissipation tests and measurements of ground water pressure. These additional measurements require a penetration interruption or may be feasible at the end of a test. It is also common to add other in situ test devices to a cone penetrometer. Table 5 presents the more common types.

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Table 5: Probes for additional in situ tests

Type of Probe	Properties	Units
Electrical Conductivity Penetrometer (ECPT)	Electrical conductivity, K	S/m
Temperature Cone Penetrometer (TCPT)	Temperature T, thermal conductivity k, volumetric heat capacity C	K, W/(m·K), MJ/m³K
Seismic Cone Penetrometer (SCPT)	S-wave velocity v_s	m/s
Cone Pressuremeter (CPMT)	Stress-strain-time response σ, ε, t	MPa, -, s
Natural Gamma Penetrometer (GCPT)	Natural gamma ray γ	CPS
Cone Magnetometer (CMMT)	Magnetic flux density B, magnetic field horizontal angle θ, vertical angle ϕ	µT, °, °
Hydraulic Profiling Tool (HPT)	Permeability k	m/s
Notes S = Siemens m = metre K = Kelvin (or °C) W = Watt J = Joule	 s = second Pa = Pascal CPS = counts per second T = Tesla	

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Soil Description and Classification

Scope

This document applies to soil description and classification as laboratory test methods, particularly:

- sample description;
- soil specimen description, whereby a specimen can be all or part of sample.

This document excludes:

- CPT-based description of soil behaviour type;
- rock description and classification;
- specific engineering geological classification systems, such as those for detailed identification of peat, chalk, and micaceous sand;
- soil stratum description, for example in the format of a geotechnical log.

Standards for Soil Description and Classification

Fugro employs a range of industry-standard systems for soil description, with additional refinements. The more important systems are:

- British Standards Institution (BSI) standard BS 5930:2015+A1:2020 (Code of Practice for Ground Investigations);
- American Society for Testing and Materials (ASTM) standards D2487-17 (Standard Practice for Classification of Soils for Engineering Purposes – United Soil Classification System) and D2488-17e1 (Standard Practice for Description and Identification of Soils – Visual-Manual Procedures);
- International Organization for Standardization (ISO) standards ISO 14688-1:2017 (Geotechnical Investigation and Testing - Identification and Classification of Soil. Part 1: Identification and Description) and ISO 14688-2:2017 (Geotechnical Investigation and Testing - Identification and Classification of Soil. Part 2: Principles for a Classification);
- International Organization for Standardization (ISO) standard ISO 19901-8:2014 (Petroleum and Natural Gas Industries - Specific Requirements for Offshore Structures. Part 8: Marine Soil Investigations).

The standards are similar, as they are (1) based on the Unified Soil Classification System (Casagrande, 1947), (2) rely on a range of relatively simple visual and manual observations, and (3) classify soils primarily according to particle size distribution and plasticity. Laboratory particle size distribution and Atterberg limits tests are used to confirm the observations. In addition, the standards include organic soils characterisation under soil particle type description.

Significant differences between the standards include the particle size boundaries and the degree to which plasticity is used as a basis for description and classification.

Where applicable, a classification system for calcareous soil can be integrated (for example based on Clark & Walker, 1977). The characteristics of calcareous soil deposits can differ substantially from those of silica-based soil deposits, primarily because of cementation and differences in void ratios.

Based on identification and description of soils, the standards given above provide a means by which soils can be classified into groups of similar composition and geotechnical characteristics.

General Procedure

The general procedure for soil description and classification is as follows:

1. Select ISO, BS, or ASTM based on local geotechnical practice or project specifications and follow the appropriate descriptive procedure; for calcareous soils, the process described by Clark & Walker (1977) may be used as an alternative.
2. Measure or estimate the particle size distribution and plasticity for use in defining the primary and secondary soil fractions.
3. Measure or estimate soil strength according to one of the following: (1) relative density of coarse soils, (2) consistency and/or undrained shear strength of fine soils, (3) cementation of cemented soils, or (4) lithification of soils undergoing diagenesis.
4. Complete the description and classification using additional terms for the soil mass characteristics and other features such as bedding, colour, and particle shape.

Soil Description and Classification using BS 5930:2015+A1:2020, ISO 14688-1:2017 and ISO 14688-2:2017

Soil Group

Soil group subdivides soils into very coarse, coarse, fine, organic, and anthropogenic soils.

Very coarse soils consist of cobbles and boulders, with particles larger than 63 mm in diameter. These soil particles are rarely sampled using standard soil sampling techniques. They are described separately, and not included when determining the proportions of the other soil components.

Characteristics of fine and coarse soils are based on particle size distribution of the coarser particles and plasticity of the finer particles. A first appraisal of physical properties is made from visual description of the soil's nature and composition, assisted by a few simple hand tests. Soils that stick together when wet and can be rolled into a thread that supports the soil's own weight (i.e. they have cohesion and plasticity) are matrix-supported and are described as fine soils. Soils that do not exhibit these properties are clast-supported and are described as coarse soils. The boundary between fine and coarse soils is on the basis of behaviour, not by weight percentage.

Organic soils contain usually small quantities of dispersed organic matter that can have a significant effect on soil plasticity and may produce a distinctive odour and have a dark grey, dark brown or dark bluish grey colour. Increasing quantities of organic matter enhance effects. Soils with a high organic content might oxidize and change colour rapidly. Organic soil descriptions in BS 5930 are based on organic content by weight determined by loss on ignition. Where organic matter is present as a secondary fraction in inorganic soil, the terms in Table 1 are used.

Soils with organic contents of up to approximately 30 % by mass and water contents of up to about 250 % behave largely as inorganic soils and are described using the terms given in Table 1. Such materials are usually transported (geologically) and would not be described as peat.

Soils comprising mainly organic materials are termed peats. They are of low density, typically 1.01 Mg/m^3 to 1.1 Mg/m^3. Peat consists predominantly of plant remains, is usually dark brown or black, and has a distinctive smell. It is generally classified according to the degree of decomposition (fibrous, pseudo-fibrous, or amorphous) and strength (firm, spongy, or plastic).

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Table 1: Description of secondary organic matter in an inorganic soil (BS 5930:2015+A1:2020)

Term	Typical Colour	Organic Content	Approximate % of dry mass
Slightly organic	Grey	Low organic content	2 to 6
Organic	Dark grey	Medium organic content	6 to 20
Very organic	Black	High organic content	> 20

The description and classification of the carbonate content of soils is made using the terms given in Table 2. The term 'non-calcareous' is not used. For the test method based on visual observations upon HCl reaction, the term 'highly calcareous' is used if no supplementary information is available.

Table 2: Carbonate content classification (based on ISO 14688-1:2017, ISO 14688-2:2017, and BS 5930:2015+A1:2020)

Term	Carbonate Content [% $CaCO_3$]	Reaction using 10 % HCl
Slightly calcareous	1 to 5	Addition of HCl produces weak or sporadic effervescence
Calcareous	5 to 25	Addition of HCl produces clear but not sustained effervescence
Highly calcareous	25 to 50	Addition of HCl produces strong and sustained effervescence
Very highly calcareous or Carbonate	> 50	

Primary Soil Fraction

Classification of coarse soils, fine soils, and composite mixtures is based both on particle size distribution and on plasticity, unless determination of plasticity is irrelevant or not feasible. Classification of fine soil is based on either particle size distribution and/or on plasticity.

Where a soil (omitting any boulders or cobbles) 'sticks together when wet, and remoulds' it is described as a fine soil ('CLAY' or 'SILT', dependent on its plasticity). When it does not stick together and remoulds, it is described as a coarse soil ('SAND' or 'GRAVEL' depending on its particle size distribution). The primary soil fraction which dominates the soil behaviour and the secondary fractions that modify that behaviour are described.

Coarse Soils

The primary soil fraction in coarse soils is sand if the dry weight of the sand fraction (0.063 mm to 2 mm particle sizes) exceeds that of the gravel fraction (2 mm to 63 mm particle sizes), and vice versa for gravel.

Particle size distribution of coarse soils can be designated based on uniformity coefficient (C_u) and coefficient of curvature (C_c) from particle size distribution curves, as presented in Table 3.

Table 3: Shape of grading curve (ISO 14688-2:2017)

Term	C_u	C_c
Uniformly graded	< 3	< 1
Poorly graded	3 to 6	< 1
Medium graded	6 to 15	< 1
Well graded	> 15	1 to 3
Gap graded	> 15	< 0.5

Sands and gravels are subdivided into coarse, medium, and fine, as defined in Table 4. Predominant size fractions are stated as, for example, 'fine and medium GRAVEL' or 'fine to coarse SAND'. The use of the conjunctions 'and' or 'to' allows differentiation between predominant fractions and a range of sizes.

Table 4: Particle size fractions of coarse soils (ISO 14688-1:2017)

Soil	Particle Size Range [mm]		
	Coarse	Medium	Fine
Gravel	63 to 20	20 to 6.3	6.3 to 2
Sand	2 to 0.63	0.63 to 0.2	0.2 to 0.063

Fine Soils

Fine soils are classified as clay or silt based on their plasticity. Onsite description of plasticity can be carried out using the terms non-plastic, low plasticity-, medium plasticity-, or high plasticity, based on hand tests. In a laboratory, plasticity can be determined based on the results of Atterberg limits tests (Table 5).

Table 5: Plasticity classification (ISO 14688-2:2017)

Term	Liquid Limit, w_L [%]
Low Plasticity	< 35
Medium Plasticity	35 to 50
High Plasticity	50 to 70
Very High Plasticity	> 70

Soils consisting solely of coarse silt may not demonstrate plasticity. They are described as silt rather than fine sand, if the grains cannot be seen with the naked eye. The distinction between clay and silt is often taken as the 'A-line', defined as $I_P = 0.73\ (w_L - 20)$, on a plasticity chart. Fine soil is classified as clay if:

$$I_P \geq 6 \text{ and } I_P \geq 0.73\ (w_L - 20)$$

where:

I_P = plasticity index [%]

w_L = liquid limit [%]

A plasticity chart may also show a 'U-line' defined (in percentages) as $I_P = 0.9\ (w_L - 8)$ and $w_L \geq 16$ %, according to Casagrande (1948). The U-line represents an approximate upper limit of correlation between plasticity index and liquid limit for natural soils.

Secondary descriptors in a fine soil may be used for materials that show behaviour that is borderline between those showing clay-like and silt-like behaviour, hence 'silty CLAY' or 'clayey SILT'. These terms are qualitative only.

Particle Shape

Description of particle shape applies to gravel, cobbles, and boulders. It typically covers the angularity of the particles (degree of rounding at edges and corners), the general form, and surface characteristics. The terms in Table 6 can be used, where appropriate.

Table 6: Designation of particle shape (ISO 14688-2:2017)

Parameter	Term
Angularity/Roundness	Very angular
	Angular
	Subangular
	Subrounded
	Rounded
	Well rounded
Form	Cubic
	Flat (or tabular)
	Elongate
Surface Texture	Rough
	Smooth

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Secondary Fractions

Table 7 presents terms for ranges of secondary fractions. If the secondary fraction is coarse, proportions are assessed by mass of different size fractions and the term 'slightly' or 'very' can precede the qualifying term.

If the secondary fraction is fine, it is identified as 'clayey' or 'silty' on the basis of its plasticity. The terms 'silty' and 'clayey' are mutually exclusive, e.g. 'gravelly clayey fine SAND'. On the other hand, the terms 'sandy' and 'gravelly' may both be used, in which case the percentages are assessed separately, e.g. 'slightly gravelly slightly sandy CLAY' means that the soil contains up to 35 % sand and up to 35 % gravel (by dry weight).

The secondary fractions as adjectives are placed with the term describing the primary fraction in the order of increasing proportion when there are two coarse soil secondary features, or coarse and then fine if one of each.

Table 7: Mixtures of coarse and fine soils (BS 5930:2015+A1:2020)

Term	Principal Soil Type	Approximate Proportion of Secondary Fraction by Mass	
		Coarse Soil	Coarse and/or Fine Soil
-	SAND and GRAVEL	About equal proportions	-
Slightly clayey or slightly silty	SAND and/or GRAVEL	-	< 5 %
Clayey or silty		-	5 % to 20 %*
Very clayey or very silty		-	> 20 %*
Slightly sandy or slightly gravelly		< 5 %	-
Sandy or gravelly		5 % to 20 %	-
Very sandy or very gravelly		> 20 %	-
Slightly sandy and/or slightly gravelly	SILT† or CLAY†	< 35 %	-
Sandy or gravelly		35 % to 65 %	-
Very sandy or very gravelly		> 65 %‡	-

Notes:
* = Or described as fine soil depending on soil behaviour
† = Can be silty CLAY or clayey SILT
‡ = Or described as coarse soil depending on assessed soil behaviour

Soil Colour

Soil colours are described using a Munsell soil colour system (e.g. Gretag-Macbeth, 2000). The Munsell colour is arranged according to three variables known as Hue, Value and Chroma. The Hue notation of a colour indicates its relation to red, yellow, green, blue, and purple. The Value notation indicates the relative lightness. The Chroma notation indicates the intensity of the colour.

Observed soil colour can change with time after sampling, for example due to changes in water content at the surface of a soil sample and due to (bio)chemical processes related to oxidation upon exposure to air.

Bedding and Interbedding

Layers of different soil types within a stratum are called bedding units. If beds of alternating or different soil types are too thin to be described as individual strata, the soil is described as interbedded or interlaminated, using the terms in Table 8, as appropriate. Where the soil types are approximately equal, 'thinly interlaminated SAND and CLAY' would, for example, be appropriate. Where one material is dominant, the subordinate material is described with a bed thickness and a bed spacing (using bedding and discontinuity spacing terms in Table 8 and Table 9 respectively), e.g. 'SAND with closely spaced thick laminae of clay'. Where two or more soil types are present in a deposit, arranged in an irregular manner, the soil is described as mixed, e.g. 'SAND with gravel size pockets (20 mm to 35 mm) of CLAY'. The spacing of sedimentary features (e.g. shell bands) and of minor structures (e.g. root holes in soils) are reported as measurements or using the spacing terms for discontinuities. These are descriptive terms that have no size connotation (e.g. pocket, lens, inclusion); where such terms are used, their size, spacing and frequency are reported.

Table 8: Bedding and interbedding thickness

Term	Thickness of Bedding Unit [mm]
Thinly (inter)laminated With† thin laminae	< 6
Thickly (inter)laminated With† thick laminae	6 to 20
Very thinly (inter)bedded‡ With† very thin beds‡	20 to 60
Thinly (inter)bedded‡ With† thin beds‡	60 to 200
Medium (inter)bedded‡ With† medium beds‡	200 to 600
Thickly (inter)bedded‡ With† thick beds‡	600 to 2 000
Very thickly (inter)bedded‡ With† very thick beds‡	> 2 000

Notes:
† = Use 'with' or other quantifying term as appropriate
‡ = Use 'bedded' or other fabric name as appropriate

Discontinuities

The term discontinuity is used to describe surfaces that separate soils of different types or form planes of weakness within the soil. Discontinuities include fissures and shear planes, and the descriptor refers to the mean spacing between such discontinuities in a soil mass. A soil is 'fissured' when it breaks into blocks along unpolished discontinuities, and 'sheared' when it breaks into blocks along polished discontinuities (which is equivalent to a slickensided soil). The spacing description (Table 9) ranges from extremely closely spaced (< 20 mm) to very widely spaced (> 2 000 mm). No other descriptive terms are used. An example would be: 'firm grey very closely fissured fine sandy calcareous CLAY with frequent silt partings'.

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The spacing terms are also used for distances between partings, isolated beds or laminae, desiccation cracks, rootlets, etc.

The surface texture of discontinuities is described, e.g. rough, smooth, or polished, as well as any colour changes or staining on discontinuities and any infilling.

Table 9: Discontinuity spacing (ISO 14688-2:2017)

Term	Spacing of Discontinuities [mm]
Extremely closely spaced	< 20
Very closely spaced	20 to 60
Closely spaced	60 to 200
Medium spaced	200 to 600
Widely spaced	600 to 2 000
Very widely spaced	> 2 000

Relative Density of Coarse Soils

Usually, soil description offers little evidence about the relative density of coarse soils. The reason for this is the substantial sampling disturbance incurred during conventional sampling operations such as push sampling, percussion sampling, and vibrocoring. Complementary investigation techniques, such as cone penetration tests (CPTs), are usually necessary. Relative density is the ratio of the difference between laboratory index void ratios of a coarse (cohesionless) soil in its loosest state and existing in situ state to the difference between its void ratios in the loosest and densest states. Loosest and densest states are relative to laboratory test methods.

Relative density is referred to in BS 5930 in terms of N-values obtained by the standard penetration test (SPT). Results of such tests may not be available. Rather than using SPT-based values, it is common practice to interpret relative density on the basis of CPT results. Ranges of relative density are given in Table 10. These ranges are in common use in the industry. They were originally presented in Lambe and Whitman (1969).

Table 10: Relative density of coarse soils

Relative Density Term	Range of Relative Density [%]
Very loose	< 15
Loose	15 to 35
Medium dense	35 to 65
Dense	65 to 85
Very dense	> 85

Consistency and Undrained Shear Strength of Fine Soils

The consistency of fine soils can be described according to Table 11. Undrained shear strength can be classified using the terms given in Table 12.

Table 11: Description of consistency (ISO 14688-1:2017)

Onsite Description Term	Definition
Very soft	Finger can be easily pushed in up to 25 mm Soil exudes between the fingers when squeezed in the hand
Soft	Finger can be pushed in up to 10 mm Soil can be moulded by light finger pressure
Firm	Thumb makes an impression easily Soil cannot be moulded by fingers, but rolls in the hand to 3 mm thick threads without breaking or crumbling
Stiff	Soil can be indented slightly by thumb Soil crumbles and breaks when rolling to 3 mm thick threads but is still sufficiently moist to be moulded to a lump again
Very stiff	Soil can be indented by thumb nail Soil cannot be moulded but crumbles under pressure

Table 12: Classification of strength (after BS 5930:2015+A1:2020)

Term Based on Measurement	Undrained Shear Strength, s_u [kPa]
Extremely low strength	< 10
Very low strength	10 to 20
Low strength	20 to 40
Medium strength	40 to 75
High strength	75 to 150
Very high strength	150 to 300
Extremely high strength	> 300

If a mineral cement appears to be present, the [nature and] degree of cementing is typically noted, e.g. 'slightly [iron oxide] cemented sand' or preferably using rock strength terms, e.g. 'very weak [carbonate] cemented SANDSTONE'.

Classification of cementation follows rock strength classification (Table 13) expressed as unconfined compressive strength σ_c:

Table 13: Cementation

Cementation	σ_c [MPa]
Slightly cemented	0.3 to 1.25
Moderately cemented	1.25 to 5.0
Well cemented	5.0 to 12.5

The term 'well cemented' in Table 13 applies to soil which also shows sublayers with little or no cementation. In case of further lithification, the soil description becomes a rock description.

Mineral Constituents and Tertiary Fractions

Mineral constituents are generally reported before the primary soil fraction, using qualitative terms such as 'slightly micaceous', 'glauconitic' or 'very shelly'. For beds of material within a soil matrix, the terminology for spacing and thickness of beds is used. For individual particles of soil or material within a soil matrix, the terms 'partings' and 'pockets' may be used.

Tertiary fractions within the soil, such as shell or wood fragments, glauconite grains, plant remains, or small soil inclusions (such as partings or pockets), can be quantified using the terms 'with rare' (< 1 % by volume), 'with occasional' (1 % to 5 % by volume), 'with frequent' (5 % to 20 % by volume), 'with numerous' (20 % to 30 % by volume), and 'with abundant' (> 30 % by volume). These terms are usually added at the end of the primary soil description (e.g. 'with frequent shell fragments', 'with rare silt pockets'). The size of the tertiary constituents can be given in mm.

Soil Odour

For anthropogenic soils, odour can be described. Terms used to describe the odour are for example 'H_2S', 'musty', 'putrid', or 'chemical'. It is emphasised that soil odour descriptions are unlikely to be fully consistent, because of factors such as variations in sample handling, ambient conditions at time of

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sample description, and strong dependence on a person's ability to detect and identify odour.

Soil Description and Classification using ASTM D2487 and ASTM D2488

The ASTM standards for soil description and classification are applicable to naturally occurring soils passing a 3 inch (75 mm) sieve. The standards identify three major soil types: coarse-grained, fine-grained, and highly organic soils. The major soil types are further subdivided into 15 specific basic soil groups.

Before a soil can be classified according to these standards, generally the particle size distribution of the minus 75 mm material and the plasticity characteristics of the minus 0.425 mm sieve material are determined.

The identification and description of silica soils in the ASTM system consists primarily of a group name and symbol, which are based on particle size distribution and plasticity, and the results of other laboratory classification tests.

Based on the results of visual observations and prescribed laboratory tests, a soil is catalogued according to the basic soil groups, assigned a group symbol(s) and name, and thereby classified.

Soil Types

The initial classification of soils as coarse-grained or fine-grained is based on percentage fines. ASTM defines the fine-coarse boundary as 0.075 mm.

A soil can be classified as coarse-grained (sand or gravel) if the percentage fines is 50 % or less. Coarse-grained soils are further classified as either sand or gravel using the results of particle size distribution analyses.

Classification of fine-grained soils (silt or clay) is based on plasticity (liquid limit and plasticity index from Atterberg limits tests).

A soil is classified as highly organic when it contains sufficient quantities of dispersed organic matter that it has an influence on the liquid limit. The soil is an organic silt or organic clay if the liquid limit after oven drying is less than 75 % of the liquid limit of the original specimen determined before oven drying.

Peat is generally classified according to the degree of decomposition and strength. When encountered, reference can also be made to the classification given in ASTM standard D4427-18.

Soil Group Name and Group Symbol

Coarse-Grained Soils

For coarse-grained soils, the dominant soil fraction is sand if the dry weight of the sand fraction, i.e. particle sizes from 0.075 mm to 4.75 mm, exceeds that of the gravel fraction, i.e. particles ranging from 4.75 mm to 75 mm, and vice versa for gravel.

Coarse-grained soils are also described as well-graded or poorly-graded based on the particle-size distribution curve, using the coefficient of uniformity (C_u) and, to a lesser extent, the coefficient of curvature (C_c) as follows:

Particle size grading of coarse-grained soils with ≤12% fines can be designated based on (C_u) and (C_c) from particle size distribution curves, as presented below:

- Sands are well-graded when $C_u \geq 6$ and C_c is between 1 and 3, and for < 5 % fines ('SW')
- Sands are poorly-graded for other values of C_u and C_c, and for < 5 % fines ('SP')
- Gravels are well-graded when $C_u \geq 4$ and C_c is between 1 and 3, and for < 5 % fines ('GW')
- Gravels are poorly-graded for other values of C_u and C_c, and for < 5 % fines ('GP').

For coarse-grained soils with fines contents > 12 %, these terms are not used. In this case, the soil is considered a coarse-grained soil with fines. The fines are determined as either clayey or silty based on the plasticity index versus liquid limit plot. Classify the soil as a clayey gravel, 'GC', or clayey sand, 'SC', if the fines are clayey. Classify the soil as a silty gravel, 'GM', or silty sand, 'SM', if the fines are silty (definition of 'clay(ey)' and 'silt(y)' as per section on fine-grained soils below).

If the fines are classified as a silty clay, 'CL-ML', classify the soil as a silty, clayey gravel, 'GC-GM', if it is a gravel or a silty, clayey sand, 'SC-SM', if it is a sand.

If the sample contains between 5 % and 12 % fines, the soil gets a dual classification using two group symbols. The first group symbol corresponds to that for a gravel or sand having < 5 % fines ('GW', 'GP', 'SW', 'SP'), and the second symbol corresponds to a gravel or sand having > 12 % fines ('GC', 'GM', 'SC', 'SM'). The group name corresponds to the first group symbol plus 'with clay' or 'with silt' to indicate the plasticity characteristics of the fines (for example: 'well-graded gravel with clay, GW-GC', 'poorly graded sand with silt, SP-SM'.

If the specimen is predominantly sand or gravel but contains 15 % or more of the other coarse-grained constituent, the words 'with gravel' or 'with sand' is added to the group name.

Sands and gravels are sub-divided into coarse, medium, and fine, as defined in Table 14.

Table 14: Size fraction descriptions for coarse-grained soils

Soil	Particle Size Range [mm]		
	Coarse	Medium	Fine
Gravel	19 to 75	-	4.75 to 19
Sand	2.0 to 4.75	0.425 to 2.0	0.075 to 0.425

Fine-Grained Soils

Fine-grained soils are classified as clay or silt according to the results of Atterberg limits tests.

The soil is an inorganic clay if the liquid limit versus plasticity index plots on or above the A-line, $I_P > 4$ %, and the presence of organic matter does not influence the liquid limit (i.e. liquid limit after oven drying is ≥ 75 % of the liquid limit of the original specimen determined before oven drying).

The soil is then further classified as lean clay if $w_L < 50$ %, and given the group symbol 'CL', or as fat clay if $w_L \geq 50$ % with group symbol 'CH'.

Soils are classified as silty clay where the liquid limit versus plasticity index plots on or above the A-line but where the plasticity index falls within the range $4 \leq I_P \leq 7$ %. Silty clays are given the group symbol 'CL-ML'.

The soil is an inorganic silt if the liquid limit versus plasticity index plots below the A-line or if $I_P < 4$ %, and the presence of organic matter does not influence the liquid limit (i.e. liquid limit after oven drying is ≥ 75 % of the liquid limit of the original specimen determined before oven drying).

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The soil is then further classified as silt if $w_L < 50$ %, and given the group symbol 'ML', or as elastic silt if $w_L \geq 50$ %, with group symbol 'MH'.

where:
I_P = plasticity index
w_L = liquid limit

If a fine-grained soil contains between 15 % and 30 % coarse material, the words 'with sand' or 'with gravel' (whichever is predominant) is added to the group name. For example, 'lean clay with sand, CL', 'silt with gravel, ML'. If the percentage of sand is equal to the percentage of gravel, use 'with sand'.

If a fine-grained soil contains > 30 % coarse material, the words 'sandy' or' gravelly' are added to the group name (whichever is predominant). For example, 'sandy lean clay, CL', 'gravelly fat clay, CH'. If the percentage of sand is equal to the percentage of gravel, use 'sandy'.

Organic Soils

If fine-grained soil has a dark colour and an organic odour when moist and warm, a second liquid limit test is performed on a test specimen which has been oven dried at 105 °C to a constant mass. For both clay and silt, or the fines component of a coarse-grained soil, the additional term organic applies if the ratio of the liquid limit of a sample (or the fines portion of the sample) after oven drying at 105 °C to the liquid limit without oven drying is less than 0.75.

Organic soils are classified in a manner similar to that for inorganic soils for plots of the liquid limit (not oven dried) versus plasticity index with respect to the A-line. Organic clays and silts with liquid limit $w_L < 50$ % are given the same group symbol 'OL'. Organic clays and silts with liquid limits $w_L \geq 50$ % are given the group symbol 'OH'.

Coarse-grained soils containing fine organic material are described using the term 'with organic fines'.

Particle Shape

The description of particle shape includes references to shape and angularity. These terms are normally used only for gravels, cobbles, and boulders, though in some cases for coarse sands.

The shape of coarse particles is described as flat, elongated, or both.

- Flat: particles with width/thickness > 3
- Elongated: particles with length/width > 3
- Flat and elongated: particles meet criteria for both flat and elongated.

Angularity terms (Table 15) are usually only applied to particles of coarse sand size and larger. A range of angularity may be stated, such as 'subrounded to rounded'.

Table 15: Angularity of coarse-grained particles

Term	Criteria
Angular	Particles have sharp edges and relatively plane sides with unpolished surfaces
Subangular	Particles are similar to angular description but have rounded edges
Subrounded	Particles have nearly plane sides but have well-rounded corners and edges
Rounded	Particles have smoothly curved sides and no edges

Secondary Constituents

Table 16 presents a summary of terms used for ranges of secondary constituents. Applicable group symbols are defined above.

Table 16: Mixtures of coarse-grained soils and fine-grained soils

Term	Principal Soil Type	Term	Approximate Proportion of Secondary Constituent	
			Coarse-Grained Soils	Fine-Grained Soils
-	SAND and/or GRAVEL*	-	-	< 5 %
-	SAND and/or GRAVEL*	with clay or silt	-	5 % to 12 %
Clayey or silty	SAND and/or GRAVEL*	-	-	> 12 %
-	SAND and/or GRAVEL*	-	< 15 % gravel or sand	-
-	SAND and/or GRAVEL*	with sand or gravel	≥ 15 % gravel or sand	-
-	SILT or CLAY	-	< 15 %	-
-	SILT or CLAY	with sand or gravel*	15 % to 29 %	-
Sandy and/or gravelly*	SILT or CLAY	-	≥ 30 %	-
Notes: * = Selection depends on which fraction has a higher percentage				

Soil Colour

Soil colours are described using a Munsell soil colour system (e.g. Gretag-Macbeth, 2000).

Structure

Criteria for describing soil structure are provided in Table 17, along with additional terms in use in the geotechnical industry.

Table 17: Criteria for describing structure

Description	Criteria
Stratified	Alternating layers of varying material or colour with the layers ≥ 6 mm thick
Laminated	Alternating layers of varying material or colour with the layers < 6 mm thick
Fissured	Breaks along definite plates of fracture with little resistance to fracturing
Slickensided	Fracture planes appear polished or glossy, sometimes striated
Blocky	Cohesive soil that can be broken down into small angular lumps which resist further breakdown
Lensed	Inclusion of small pockets of different soils, such as small lenses of sand scattered through a mass of clay
Homogeneous	Same colour and appearance throughout

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Description	Criteria
Gassy*	Soil has a porous nature and there is evidence of gas, such as blisters
Expansive*	Visibly expands after sampling
Platy*	A stratified appearance when the soil can be broken into thin horizontal plates
Cemented*	Material grains bound together forming an intact mass
Notes: * = not part of ASTM D2488	

The distance between the fissures, shear planes, and expansion cracks is noted using the terms in Table 9.

Consistency and Undrained Shear Strength of Fine-Grained Soils

The consistency of fine soils can be assessed on site, in case of no onsite laboratory. Table 18 shows terms for the designation of consistency of fine-grained soils in accordance with the results of manual tests.

Table 18: Criteria for describing consistency

Description	Criteria
Very soft	Thumb will penetrate soil more than 25 mm
Soft	Thumb will penetrate soil about 25 mm
Firm	Thumb will indent soil about 6 mm
Hard	Thumb will not indent soil but readily indented with thumbnail
Very hard	Thumbnail will not indent soil

Descriptions of undrained shear strength of fine-grained soils are not part of the ASTM classification system. If required by project specifications, undrained shear strength ranges according to Table 19 can be presented.

Table 19: Undrained shear strength of fine-grained soils

Term	Undrained Shear Strength*	
	[kPa]	[ksf]†
Very soft	< 12.5	< 0.25
Soft	12.5 to 25	0.25 to 0.50
Firm	25 to 50	0.50 to 1.0
Stiff	50 to 100	1.0 to 2.0
Very stiff	100 to 200	2.0 to 4.0
Hard‡	200 to 400	4.0 to 8.0
Very hard‡	> 400	> 8.0

Notes:
* = From Terzaghi and Peck (1967)
† = ksf used primarily for US projects
‡ = Upper boundary for 'hard', and a range for 'very hard' have been added

Written Soil Descriptions

In a soil description, the main characteristics are typically given using the following standard word sequence, as applicable:

1. Relative density/consistency/undrained shear strength
2. Discontinuities
3. Bedding
4. Colour
5. Composite soil types: particle size distribution and grading, shape and size
6. Tertiary fractions either before or after the primary soil fraction as appropriate
7. PRIMARY SOIL FRACTION, based on grading and plasticity

For example: Firm closely fissured dark olive grey sandy calcareous CLAY with few silt pockets.

Description of Carbonate Soils using Clark & Walker (1977)

For carbonate soils, the classification system by Clark and Walker (1977) may be used to describe the carbonate content, particle size, and degree of induration.

Particle Size

Clark & Walker (1977) follows approximately the same boundaries between clay, silt, sand and, gravel fractions as used in BS 5930:2015+A1:2020 and ISO 14688-1:2017.

Carbonate Content

Carbonate content is used for non-indurated (unconfined compressive strength < 300 kPa) carbonate soils in the Clark & Walker classification system (Table 20). The description method does not distinguish between types of carbonate material and assumes that non-carbonate particles are siliceous.

Induration

Induration is the process of hardening of sediments through cementation and/or compaction. Degree of induration is classified by Clark & Walker (1977) by means of unconfined compressive strength. In Table 20, terms for non-indurated (unconfined compressive strength σ_c < 0.3 MPa) carbonate deposits are given, and in Table 21 for slightly (σ_c = 0.3 MPa to σ_c = 12.5 MPa) and moderately indurated (σ_c =12.5 MPa to σ_c = 100 MPa) carbonate deposits.

Table 20: Description and classification of non-indurated carbonate soils (based on Clark & Walker, 1977)

Fine Soils		Coarse Soils		Carbonate Content (% $CaCO_3$)
CLAY	SILT	Silica SAND	GRAVEL	< 10
Calcareous CLAY	Calcareous SILT	Calcareous silica SAND	Mixed carbonate and non-carbonate GRAVEL	10 to 50
Carbonate CLAY	Siliceous carbonate SILT	Siliceous carbonate SAND		50 to 90
Carbonate CLAY	Carbonate SILT	Carbonate SAND	Carbonate GRAVEL	> 90

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Table 21: Description and classification of slightly to highly indurated carbonate soils and rocks (based on Clark & Walker, 1977)

Grain Size of Particulate Deposits				Carbonate Content [% $CaCO_3$ of soil dry mass]	σ_c [MPa]
< 0.002 mm	0.002 mm to 0.06 mm	0.06 mm to 2 mm	2 mm to 60 mm		
Slightly Indurated					
CLAYSTONE	SILTSTONE	SANDSTONE	CONGLOMERATE or BRECCIA	< 10	0.3 to 12.5
Calcareous CLAYSTONE	Calcareous SILTSTONE	Calcareous SANDSTONE	Calcareous CONGLOMERATE	10 to 50	
Clayey CALCILUTITE	Siliceous CALCISILTITE	Siliceous CALCARENITE	Conglomeratic CALCIRUDITE	50 to 90	
CALCILUTITE	CALCISILTITE	CALCARENITE	CALCIRUDITE	> 90	
Moderately Indurated					
CLAYSTONE	SILTSTONE	SANDSTONE	CONGLOMERATE or BRECCIA	< 10	12.5 to 100
Calcareous CLAYSTONE	Calcareous SILTSTONE	Calcareous SANDSTONE	Calcareous CONGLOMERATE	10 to 50	
Fine-grained argillaceous LIMESTONE	Fine-grained siliceous LIMESTONE	Siliceous detrital LIMESTONE	Conglomeratic LIMESTONE	50 to 90	
Fine-grained LIMESTONE		Detrital LIMESTONE	Conglomerate LIMESTONE	> 90	

The Clark & Walker system does not include reef limestone (biolithite). Reef limestone represents an in situ accumulation of biological origin (e.g. coral reef) and consists largely of carbonate skeletal material of colonising organisms. The carbonate content normally exceeds 90 %. Classification of strength follows rock description procedures.

Particulate Deposits

The geological origin of a single particle type allows the following descriptions (optional).

Clastic

Sediment transported and deposited as grains of inorganic origin. Typical clastic particles are:

- quartz grains: clear or milky white and ranging from very angular to very rounded; commonly a frosted surface for wind-blown grains;
- feldspar grains: varying in colour from milky white to light yellowish brown;
- mica flakes: varying in colour from gold-coloured to dark brown;
- dark mineral grains: usually of igneous or metamorphic origin with undetermined mineralogy;
- silicate grains: undetermined mineralogy;
- rock fragments: including fragments of carbonate rock;
- debris: deposit of rock fragments of a variety of particle sizes which may include sand and finer fractions; typical examples are rock debris and coral debris.

Organic

Remains of plants and animals that consist mainly of carbon compounds.

Bioclastic

Sediment transported and deposited as grains of organic origin. Examples of bioclastic particles are:

- Calcareous algae: crustal or nodular growths or erect and branching forms produced by lime-secreting algae; microstructures include layered, rectangular structures and internal fine tube-like structures;
- Foraminifera: hard sediment test (external skeleton) consisting of calcite or aragonite and produced by unicellular organisms; commonly less than 1 mm in diameter, multi-chambered and intact;
- Sponge spicules: spicules of siliceous sponges in a variety of rayed shapes; dimensions ranging from less than 1 mm to over 1 cm in length but usually less than 1 mm in width;
- Corals: commonly consisting of small fibres set perpendicular to the walls and septal surfaces; mainly aragonite composition for relatively recent forms; conversion of aragonite to calcite for earlier corals, usually with consequent loss of original structural details;
- Echinoids: hard part of echinoids consisting of a plate or skeletal element forming a single crystal of calcite; five-rayed internal symmetry for spines of echinoids; typical widths ranging from several mm to a few cm;
- Bryozoans: chambered cell-like structures that are considerably coarser than those of calcareous algae; either aragonite or calcite composition; possible cell in-fill consisting of clear calcite and/or micrite;
- Bivalves and gastropods: mollusk shells, chiefly of aragonite composition; inner layer of aragonite protected by an outer layer of calcite for some bivalve shells and gastropods.

Oolitic

Sediment consisting of solid, round, or oval, highly polished and smooth coated grains, which may or may not have a nucleus. The coating consists of chemically precipitated aragonite, possibly converted to calcite. Oolites have concentric structures and may also have radial structures. The grains are generally less than 2 mm diameter.

Pelletal

Sediment consisting of well-rounded grains of ellipsoidal shape and no specific internal structure. The composition is clay to silt-sized carbonate material, which is probably the excretion product of sediment-eating organisms. Pellets may have an oolitic crust. The grains are generally less than 2 mm diameter.

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Structure of Non Particulate Deposits

Reef

Soil or rock formed by in situ accumulation or build-up of carbonate material by colonial organisms such as polyps (coral), algae (algal mats or balls) and sponges.

Orthochemical

Orthochemical components precipitated during or after deposition. These components can include: (1) pyrite spherulites and grains, (2) crystal euhedra of anhydride or gypsum, (3) replacement patches and nodular masses of anhydrite and gypsum. Single grains are rare.

Geological Information

Specific geological terms can assist the geotechnical soil description by providing information on stratigraphy, origin (genesis) or regional significance (optional). Examples are:

- Time stratigraphy, such as Eemian and Pleistocene;
- Lithostratigraphy, such as Yarmouth Roads Formation;
- Depositional environment, such as marine, glacio-lacustrine, and residual soil;
- Regional significance, such as chalk and mud.

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Geotechnical Laboratory Tests

Scope

This document summarises geotechnical laboratory test methods for soil. Some of the test methods also apply to rock as defined by ISO 14689:2017.

Fugro strives to arrange testing in registered laboratories with formal accreditation to ISO/IEC 17025:2017 or equivalent standards. This document summarises test methods used by many Fugro geotechnical laboratories. Test methods used by onsite laboratories are often identical or generally equivalent.

Laboratory tests are carried out in general accordance with standards published by ASTM International (ASTM), British Standards Institution (BSI), and International Organization for Standardization (ISO). Note that ISO 19901-8:2014 refers mainly to test procedures published in other documents, with some additional recommendations. In-house test procedures adopted for some tests are generally based on published recommendations for which no standards are available. Detailed work instructions and calibration details are available for inspection at the laboratory.

The focus of this document is on specific laboratory test methods. Some geotechnical parameter values can be indirectly derived, for example degree of saturation and sensitivity. These indirect methods are listed as test method, with descriptions given under the first appropriate heading.

Some of the laboratory tests allow various optional procedures. These procedures are not applicable, unless specifically agreed.

Laboratory test results show depth defined as vertical distance between ground surface or seafloor and top of the laboratory test specimen, unless indicated otherwise.

Procedure

A programme of laboratory testing is performed according to project specifications. Commonly, the project specifications are part of procedures for site investigation and site characterisation, including sampling and sample handling. Depending on the project specifications, the following steps apply to laboratory testing:

1. Prepare a laboratory test strategy;
2. Perform sampling, including sample handling;
3. Prepare a laboratory test programme, including a pre-test applicability assessment;
4. Select sample(s) with sufficient dimensions and mass for performance of a test or a test series;
5. Assess the feasibility and applicability of a test by sample inspection and, if required, selection of alternative sample(s) if adequate completion of a test or test series proves impracticable;
6. Assign tests to specific samples or parts of samples;
7. Where applicable, determine test conditions according to the laboratory test strategy;
8. Select suitable test apparatus, including check of calibration certificate(s) where applicable;
9. Where applicable, perform sample batching to obtain an adequate quantity of material for preparation of one test specimen or for multiple specimens for a test series;
10. Prepare test specimen;
11. Perform pre-test applicability assessment and revise the laboratory test programme where required;
12. Perform test according to the project specifications and within applicability of the test type and/or method;
13. Perform post-test applicability assessment and revise the laboratory test programme where required;
14. Process test results including assessment against quality control procedures;
15. Report test results according to project specifications, test methods and standards.

The steps of the procedure are broadly successive. Loops in the procedure can be introduced depending on e.g. (1) a laboratory test strategy requiring results from a particular test as input for a subsequent test and (2) the results of applicability assessment.

Sampling and Sample Handling

The feasibility of a particular laboratory test relates to the sampling practice and sample handling for a particular soil and depends on factors such as soil type, available amount of sample material, and sample quality.

ISO 22475-1:2021 includes a classification system for five quality classes of soil samples, aligned to five categories of sampling methods. The quality classes consider feasibility of specific geotechnical identification and laboratory tests. A summary of these quality classes is as follows:

Quality Class 1:	strength, stiffness, and compressibility
Quality Class 2:	permeability, unit weight, boundaries of strata - fine
Quality Class 3:	water content
Quality Class 4:	particle size analysis, Atterberg limits, boundaries of strata - broad
Quality Class 5:	soil type and sequence of layers

The higher class includes the laboratory tests of the lower class. The alignment with the five sampling categories is such that Sampling Category A enables samples of Quality Classes 1 to 5 to be obtained and Sampling Category E enables only Quality Class 5 to be obtained.

Sample handling can include multiple stages of sample packaging and unpacking. These stages are typically related to loops in the general procedure and to sample storage requirements. Logistics can apply for transfers of samples between a laboratory and a sample storage facility.

Soil parameter values can vary with temperature. Tests are generally conducted at laboratory temperatures of around 20 °C. Any tests conducted at specific temperatures and any corrections for temperature are explicitly reported.

Test references: ISO 22475-1:2021, BS 5930:2015+A1:2020, ISO 19901-8:2014

Sample Batching

The laboratory test strategy can include sample batching of coarse soil. The purpose of sample batching is to allow specimen preparation for two or more tests on material representative for the batch. Sample batching allows comparison and integration of multiple laboratory test results for estimation of geotechnical characteristics of a particular ground type.

Test reference: ISO 19901-8:2014

Applicability Assessment

Test standards typically provide explicit and indirect indications of applicability of a test method. Laboratory testing can include

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applicability assessment performed during a pre-test phase or post-test phase. Usually, a reasonable estimate of test feasibility is possible at the time of sampling. A further refinement of applicability is possible in the laboratory prior to the primary test phase (pre-test phase) and, in some cases, only after the primary test phase (post-test phase). Applicability assessment may lead to one or more of the following outcomes: (1) reporting of test results, with or without comments on applicability, (2) rejection of a sample or specimen for testing upon inspection or other observations made in the laboratory, (3) early termination of testing, with no results from the primary test phase and (4) selection of an alternative sample or test specimen.

A test stage of soil description and imaging can be required for applicability assessment. Some laboratory test programmes require input from specific tests for applicability assessment.

In practice, laboratory tests may be performed outside of the intended applicability of the test method. Typically, such applicability situations are not explicitly reported.

An example of an explicit indication of applicability would be that the test method is intended for an intact uniform homogeneous clay. Note that no soil sample can be taken from the ground in a perfectly undisturbed state. ISO 22475-1:2021 lists more than 15 reasons for soil sample disturbance that can affect intact sample quality. Indirect indications of applicability can consist of e.g. a low-stress limit for equipment capability.

Test references: ISO 22475-1:2021, ISO 19901-8:2014

Specimen Preparation

Specimen preparation depends on specimen type required for a particular test method. A particular test method can include multiple options for specimen preparation. Examples are as follows:

- Disturbed specimen;
  - Preparation typically consists of selecting the required quantity or mass of soil from a sample;
  - Preparation can include removal of a soil fraction, for example particle size $D > 0.425$ mm and can include breaking down of aggregations of particles;
  - Preparation can include drying, air drying, or oven drying;
- Intact (undisturbed) specimen;
  - Preparation typically consists of trimming to a required geometry, for example by using trimming tools or a cutting ring;
  - Preparation can include the use of support devices for soil susceptible to collapse or slumping and the use of fill-in material in case of small gaps or voids in a specimen;
- Compacted specimen;
  - Preparation typically consists of (1) applying a specified compacting energy at a specified water content or (2) achieving a specified dry density at a specified water content;
  - Methods for compaction include tamping, kneading, ramming, vibration, and static compression;
  - Preparation typically includes the use of a mould;
- Reconstituted specimen – coarse soil;
  - Preparation typically consists of placing dry, moist, or wet soil in a mould, using one of the following methods: air pluviation, moist tamping or water pluviation.
  - A further stage of specimen densification can be applied, for example by one of the compaction methods listed for compacted specimen. Cyclic compression and cyclic preshearing can also be applied;
- Reconstituted specimen – fine soil;
  - Preparation typically consists of (1) mixing soil with water to a slurry, poured into a consolidation cell and (2) allowing consolidation by application of static or monotonic consolidation stresses and (3) removal of the specimen from the consolidation cell for trimming as for an intact specimen;
- Remoulded specimen;
  - Preparation typically consists of mixing, squeezing, and kneading of a sample with minimum change in water content;
  - Remoulded soil can be placed into a mould and can be trimmed to a specified geometry;
  - A waiting time can be applied for allowing soil set-up effects to take place.

Inevitably, each of the specimen preparation methods has geotechnical constraints within the context of the selected test method.

Test reference: ISO 19901-8:2014

Specimen Saturation

Specimen saturation can be important for test methods requiring soil samples meeting Quality Class 1 and Quality Class 2.

Some test methods incorporate specific procedures for achieving specimen saturation, for example methods for consolidated undrained triaxial testing.

Some test methods aim for achieving specimen saturation, for example by careful sample handling, application of external pressure, deairing and submerging a specimen in water. Examples for this category include the unconsolidated undrained triaxial test and the constant head permeability test by permeameter. Saturated conditions may or may not be achieved. In most cases, any effects on test results will remain unknown. Publications may be available on possible effects on test results.

Test references: ISO 22475-1:2021, BS 5930:2015+A1:2020, ISO 19901-8:2014

Intact Sample Quality

Intact Sample Quality - $\Delta e/e_0$

The $\Delta e/e_0$ test method for indication of intact (undisturbed) sample quality considers a specific test stages, based on re-compression of a test specimen in oedometer, direct simple shear or triaxial apparatus. The parameter Δe represents the change in void ratio Δe from an initial laboratory value (e_0) of a prepared specimen at atmospheric conditions to the specimen void ratio upon re-compression to estimated in situ effective stress conditions (σ'_{v0} and σ'_{h0}), where σ'_{v0} is the effective in situ vertical stress and σ'_{h0} is the effective in situ horizontal stress.

Table 1 from ISO 19901-8:2014 presents a classification scheme using $\Delta e/e_0$. The presented criteria are based on tests on marine clays acquired onshore in the depth range 4 m to 25 m, with plasticity index in the range 6 % to 43 %, water content 20 % to 67 % and overconsolidation ratios of 1 to 4. The criteria must be used with caution for soils outside this range.

Table 1: Intact sample quality - $\Delta e/e_0$

Over-consolidation Ratio	$\Delta e/e_0$			
	1 (very good to excellent)	2 (good to fair)	3 (poor)	4 (very poor)
1 to 2	< 0.04	0.04 to 0.07	0.07 to 0.14	> 0.14
2 to 4	< 0.03	0.03 to 0.05	0.05 to 0.10	> 0.10

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The $\Delta e/e_0$ criteria represent a simplification, as they ignore important soil changes during the process of sampling and sample handling up to specimen preparation in a geotechnical laboratory. This simplification avoids interpretation anomalies related to uncertainties in laboratory values for soil unit weights, water contents and density of solid particles.

Some of the laboratory test methods imply limited or no control on effective stress conditions. Such limited control is primarily related to uncertainty on specimen saturation and largely unknown lateral confinement of the test specimen. This can affect calculated values of $\Delta e/e_0$.

The $\Delta e/e_0$ criteria assume no-gas within the pore water. Gas can cause an increase in void ratio when recovering samples to surface. The result is a correspondingly larger change in void ratio when returning a specimen back to the estimated effective stress conditions in situ. In such case, it is likely that the undrained shear strength would be less affected than soil stiffness, as the void ratio in situ has been partially restored. Changes in soil fabric remain.

Values for $\Delta e/e_0$ should exclude secondary consolidation. In practice, no correction for secondary consolidation will be applied. This practice underestimates intact sample quality, particularly for incremental loading oedometer tests with 24-hour load increments and longer.

Intact Sample Quality - ε_v

The ε_v (volumetric strain) method is similar to the $\Delta e/e_0$ method. Values of ε_v represents volumetric strain derived from recompression of a prepared specimen at atmospheric conditions to estimated in situ effective stress conditions (σ'_{v0} and σ'_{h0}), where σ'_{v0} is the effective in situ vertical stress and σ'_{h0} is the effective in situ horizontal stress.

Table 2 presents a classification scheme using ε_v according to Terzaghi et al. (1996). The criteria apply to clays with an overconsolidation ratio of less than about 3 to 5.

Parameters such as effective preconsolidation pressure σ'_p and undrained shear strength s_u preferably require laboratory specimen with SQD equal to B or better (DeGroot et al., 2005).

Table 2: Intact sample quality - SQD

Volumetric Strain ε_v [%]	SQD
< 1	A
1 to 2	B
2 to 4	C
4 to 8	D
> 8	E
Note: SQD: Sample Quality Designation	

Comments about applicability and constraints for the $\Delta e/e_0$ method also apply to the ε_v method.

Soil Description and Imaging

Soil description typically considers (1) the Unified Classification System (Casagrande, 1947), (2) a range of relatively simple visual and manual observations, and (3) classification of soil according to particle size distribution and plasticity.

Soil description can include imaging of a ground sample or specimen by photography, laser scanning and acoustic methods.

The level of detail and accuracy in soil description and interpretation depend on factors such as test data, sample size, quality, coverage, availability of supplementary information, and project specifications.

Test references: ASTM D2487-17e1, ASTM D2488-17e1, BS 5930:2015+A1:2020, ISO 14688-1:2017, ISO 14688-2:2017, ISO 19901-8:2014

Geotechnical Index Testing

Water Content

The water content is determined by drying selected moist/wet soil material for at least 18 hours to a constant mass in a 110 °C drying oven. The difference in mass before and after drying is used as the mass of the water in the test material. The mass of material remaining after drying is used as the mass of the solid particles. The ratio of the mass of water to the measured mass of solid particles is the water content of the material. This ratio can exceed 1 (or 100 %).

Test references: ASTM D2216-19, ISO 17892-1:2014, ISO 17892-1:2014/AMD 1:2022, ISO 19901-8:2014

Unit Weight – Volume-Mass Calculation

Measurement of volume and mass of a soil sample allows calculation of unit weight (γ). For fine (cohesive) soils, a soil specimen is generally obtained from a standard steel cylinder with cutting edge, which is pushed manually into the extruded soil sample. Preference is given to a 100 ml cylinder (area ratio of 12 %), but a volume of 33.3 ml (area ratio of 21 %) may be used when insufficient homogeneous sample is available. If possible, a specimen of coarse (non-cohesive) soil is obtained by selecting a part of a cylindrical soil sample, trimming the end surfaces, and measuring height and diameter. This method also applies to specimens of fine soil selected for strength and/or stiffness (e.g. triaxial and oedometer) tests.

Unit weight γ refers to unit weight of the soil specimen at the water content at the time of test.

The method excludes correction for pore water salinity r (contains dissolved solids), in situ pressure and temperature. Figure 1 provides an indication of error in calculated submerged unit weight γ' versus submerged unit weight corrected for salinity, γ'^* (Kay et al., 2005). Typical seawater salinity is 35 g salt per kg seawater (r = 0.035). Correction for salinity is optional.

Optionally, unit weight (γ_d) can be calculated from the mass of oven-dried soil and the initial specimen volume.

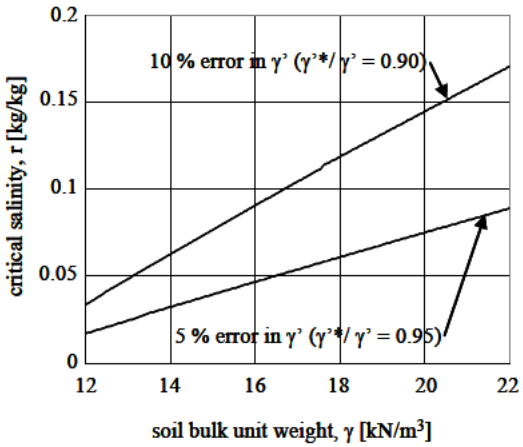

Figure 1: Soil unit weight versus pore water salinity

Test references: ASTM D7263-21, ISO 17892-2:2014, ISO 19901-8:2014

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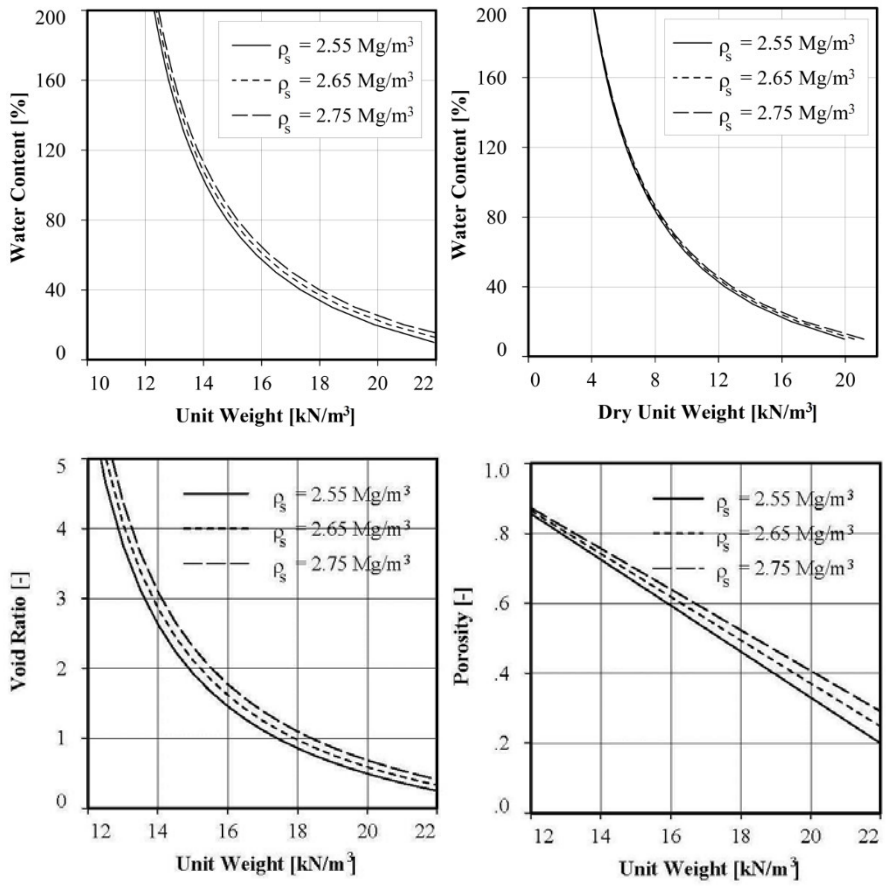

Figure 2: Influence of density of solid particles on soil unit weight

Unit Weight – Derived from Water Content

Water content (w) allows estimation of soil unit weight (γ) and unit weight of dry soil (γ_d) if specimen saturation by pore water is presumed. This practice requires input on density of solid particles (ρ_s).

Figure 2 shows water content versus (dry) unit weight presuming specimen saturation with non-saline pore water. Correction for (high) pore water salinity (contains dissolved solids) is optional.

Test reference: In-house

Void Ratio and Porosity

Void ratio (e) and porosity (n) can be derived from soil unit weight (γ). This practice requires input on density of solid particles (ρ_s).

The presented diagrams showing void ratio (porosity) versus unit weight presume saturation of non-saline pore water.

Test reference: In-house

Degree of Saturation

Degree of saturation (S_r) can be derived from porosity (n) and water content (w). Theoretical values of S_r are between 0 and 1 (or 0 % and 100 %). Inevitable test uncertainties for n and w can result in:

- $S_r > 1$;
- $S_r < 1$ for a saturated test specimen.

Test reference: In-house

Density of Solid Particles – Conventional Pycnometer

The density of the solid particles of an oven-dried soil sample is determined by means of a stoppered-bottle pycnometer, using distilled water. The method is considered applicable to solid particles that are not soluble in water. For soils with a high organic content, a different liquid may be selected.

Soils with high pore-water salinity (contain dissolved solids) require use of a gas pycnometer. This is optional.

Test references: ASTM D854-14, ISO 17892-3:2015, ISO 19901-8:2014

Particle Shape – Descriptive

Particle shape of coarse soil can be determined by visual comparison of, for example, particle roundness and sphericity with standard particle shapes. Visual observations can be supported by microscopy and photographs. The standard shapes are typically presented together with the test results.

Test references: ISO 9276-6:2008, ISO 14688-2:2017, ISO 19901-8:2014, Krumbein & Sloss (1963)

Particle Shape – Image Analysis

Analysis of digital images can provide particle shape expressed by shape coefficients, for example, for particle roundness and sphericity. The applicability of the method depends on equipment type. For geotechnical purposes, optical equipment is common, covering particle sizes ranging from about 0.04 mm to 20 mm. Specimen preparation includes methods for particle dispersion, where applicable.

Test references: ISO 9276-6:2008, ISO 14688-2:2017, ISO 19901-8:2014

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Particle Size Analysis – Sieving and Sedimentation

Particle size analysis can be performed by means of sieving and/or sedimentation methods. Sieving is carried out for particles that would be retained on a 0.063 mm (ISO and BS) or 0.075 mm (ASTM) sieve, while additional sedimentation analysis may be carried out when a significant fraction of the material passes a 0.063 mm (0.075 mm) sieve.

In a sieve analysis, the mass of soil retained on each sieve is determined and expressed as a percentage of the total mass of the sample. Prior to sieving, samples are treated with a dispersing agent (sodium hexametaphosphate), rinsed on a 0.063 mm (0.075 mm) sieve and dried.

Sedimentation methods (using hydrometer or pipette) allow measurement of the density of a suspension consisting of fine soil particles and distilled water, to which a dispersion agent is added. This suspension is mixed using a high-speed stirrer. Testing is performed in a thermostatically controlled water bath (25 °C ± 0.5 °C). The particle size is calculated according to Stokes' Law for a single sphere, on the basis that particles of a particular diameter were at the surface of the suspension at the beginning of sedimentation and had settled to the level at which the hydrometer or pipette is measuring the density of the suspension. These calculations require a value for the density of solid particles. Generally, a value of 2.65 Mg/m^3 is assumed. When other values are used, this is included in the laboratory report. Sedimentation results for selected particle sizes are presented as a percentage of the total mass of the soil sample.

Particle size is presented on a logarithmic scale so that two soils having the same degree of uniformity are represented by curves of the same shape regardless of their positions on the particle size distribution plot. The general slope of the distribution curve may be described by the coefficient of uniformity C_U, where $C_U = D_{60}/D_{10}$, and the coefficient of curvature C_C, where $C_C = (D_{30})^2/D_{10} \times D_{60}$. D_{60}, D_{30}, and D_{10} are effective particle sizes indicating that 60 %, 30 %, and 10 % respectively of the particles (by weight) are smaller than the given effective size.

Combined presentation of results from sieving and sedimentation methods requires data harmonising in the area of overlap, i.e. near the 0.06 mm particle size.

Test references: ASTM D6913/D6913M-17, ASTM D7928-21e1, ISO 17892-4:2016, ISO 19901-8:2014

Particle Size Analysis – Image Analysis

Particle size analysis can be performed by analysis of digital images obtained by optical or electron detection systems. Particle size range depends on equipment type. A single instrument can cover common particle sizes, e.g. from 0.04 mm to 20 mm.

Specimen preparation includes methods for particle dispersion, where applicable.

Data presentation is as for 'Particle Size Distribution – Sieving and Sedimentation'. Image analysis for particle size can be integrated with (1) supplementary methods for particle size analysis, such as percentage fines and sedimentation methods, and (2) data that are additional to particle size distribution, e.g. particle shape.

Test references: ISO 13322-1:2014, ISO 13322-2:2021, ISO 19901-8:2014

Percentage Fines

The percentage fines test identifies the proportions of fine (< 0.063 mm for BS/ISO and < 0.075 mm for ASTM) and coarse (> 0.063 mm) particle sizes of a soil sample by wet sieving through a 0.063 mm (0.075 mm) sieve. Prior to sieving, the sample is treated with a dispersing agent. The percentage fines $FC_{0.063}$ ($FC_{0.075}$) is defined as the ratio of dry mass of soil passing the 0.063 mm (0.075 mm) sieve to the dry mass of the total soil sample, expressed as a percentage.

Test references: ASTM D1140-17, ISO 17892-4:2016, ISO 19901-8:2014

Atterberg Limits

Atterberg limits are determined on soil specimens with a particle size of less than 0.425 mm. If necessary, coarser material is removed by dry sieving. The Atterberg limits refer to arbitrarily defined boundaries between the liquid and plastic states (liquid limit, w_L), and between the plastic and brittle states (plastic limit, w_P) of fine soils. They are expressed as water content, in percentage. The range of water contents over which a soil behaves plastically is the plasticity Index, I_P. This is the difference between the liquid limit and the plastic limit ($w_L - w_P$).

The liquid limit is defined as the water content at which a penetration of 10 mm or 20 mm of a standard fall cone into soil placed in a standard cup is reached (penetration depending on type of fall cone used). Alternatively, the liquid limit can be defined as the water content at which a part of soil placed in a standard cup and cut by a groove of standard dimensions will flow together at the base of the groove when the cup is subjected to 25 standard shocks. The four-point liquid limit test is usually carried out. Distilled water may be added during soil mixing to achieve the required consistency.

The plastic limit is defined as the water content at which a soil can no longer be deformed by rolling into 3 mm diameter threads without crumbling.

Atterberg limits can be presented on a plasticity chart (liquid limit versus plasticity index). BS, ISO, and ASTM standards define plasticity classifications (e.g. low, high).

Test references: BS 5930:2015+A1:2020, ASTM D2487-17e1, ASTM D4318-17e1, ISO 19901-8:2014, ISO 14688-2:2017, ISO 17892-12:2018, ISO 17892-12:2018/AMD 1:2021, ISO 17892-12:2018/AMD 2:2022

Minimum Index Dry Density

The minimum index dry density ($\rho_{\mathrm{d;min}}$) of cohesionless soil is determined from the mass of oven-dry material that is deposited by slowly withdrawing a soil-filled funnel from a standard mould.

Test reference: ASTM D4254-16, Knudsen et al. (2020)

Maximum Index Dry Density – Impact Compaction

The maximum index dry density ($\rho_{\mathrm{d;max}}$) of cohesionless soil is determined from the mass of oven-dry, compacted soil in a standard mould (Table 3). The soil is compacted in five layers, with each layer being subjected to respectively 5, 10, 20, 40 and 80 blows from a standard, hand-held hammer.

Preference is given to the large mould; actual application depends on size of sample available.

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Table 3: Test details for maximum index dry density – impact compaction

Equipment		70.5 ml mould	554 ml mould
Hammer mass	[g]	185	750
Drop height	[mm]	300	390
Cross-sectional area	[mm²]	1006	38 500

Test reference: DGF (2001)

Maximum Index Dry Density – Vibrating Table with Surcharge

The maximum index dry density ($\rho_{d;max}$) of cohesionless soil is determined from the mass of submerged and compacted soil in a standard mould which is mounted on a 'sieve-shaker'. The soil is compacted in one layer using a 2 mm vertical peak-to-peak amplitude and a vibration time of 2 min without a surcharge, followed by 15 s with a 7 kPa surcharge. Two tests are performed per scheduled sample and the average value of $\rho_{d;max}$ of the two tests is reported as the test result.

Test reference: Knudsen et al. (2020)

Maximum Index Dry Density – Vibrating Hammer

The maximum index dry density ($\rho_{d;max}$) is obtained by compacting soil that has been passed through a 4 mm sieve into a mould at a range of water contents. The first sample is thoroughly mixed with water, to produce a soil with a 4 % water content, and then compacted in three equal layers using a vibrating hammer for a period of 10 seconds per layer. The top section of the mould is removed, and the sample levelled in the bottom section of the mould. The unit weight of the sample is calculated, and a representative portion of soil is removed for water content determination.

The test is repeated at four further water contents. By determining the dry unit weight achieved at each water content, a maximum dry unit weight may be estimated. There is evidence of breakdown of crushable particles during a test.

Equipment dimensions are as follows:

- Volume of mould: 96.4 ml
- Hammer: Milwaukee heavy duty 545S
  - 1300 W nominal / 650 W release
  - rotation/min: 300
  - hammer force: 8.5 J
  - mass: 6.7 kg

Test reference: In-house

Geochemical Testing

Organic Matter Content – Dichromate Oxidation

An oven-dried (50 °C) soil sample is mixed with potassium dichromate solution and left for 30 min to allow the oxidation of organic matter to proceed. The solution is titrated with a ferrous sulphate solution (to determine the amount of excess potassium dichromate). The organic matter content is defined as the ratio of the total volume of potassium dichromate solution used to oxidize the organic matter in the soil sample to the mass of the initial dried soil sample (Walkley and Black's method). It is expressed as a percentage.

Note: soils containing sulphides or chlorides have been found to yield inaccurate (too high) organic matter content values using this procedure.

Test reference: BS 1377-3:2018

Organic Matter Content – Loss on Ignition

An oven-dried (105 °C) soil sample is heated to 440 °C, until the soil mass is constant. The mass is measured before and after heating. The organic matter content is defined as the ratio of the mass loss due to heating to the original mass of the dried soil sample and is expressed as a percentage.

Note: the mass loss on ignition test is reliable for (1) sandy soils that contain little or no clay and no carbonate and (2) peats and organic clays containing more than 10 % organic matter.

Test references: BS 1377-3:2018, ASTM D2974-20e01, NEN 5754:2014, ISO 19901-8:2014

Carbonate Content – Classification of Results

Carbonate content ($CaCO_3$) is expressed as a percentage of dry mass of the original soil. Classification of carbonate content results can be done by using one or multiple laboratory test methods. Results of multiple test methods can be combined and classified using the terms given in Table 4. Comments are as follows.

- ISO 14688-1:2017 and ISO 14688-2:2017 consider an additional term 'non-calcareous' for a carbonate content of < 1 %, not shown in the presented table.
- The applicability range for the test method 'carbonate content – HCl reaction' (below) excludes the classification 'very highly calcareous or carbonate' (> 50%). This means that results of this method (addition of HCl produces strong and sustained effervescence) are attributed to 'highly calcareous' yet can also mean 'very highly calcareous or carbonate'.

Table 4: Test details for carbonate content – classification of results

Term	Carbonate Content [% $CaCO_3$]	Reaction using 10 % HCl
Slightly calcareous	1 to 5	Addition of HCl produces weak or sporadic effervescence
Calcareous	5 to 25	Addition of HCl produces clear but not sustained effervescence
Highly calcareous	25 to 50	Addition of HCl produces strong and sustained effervescence
Very highly calcareous or Carbonate	> 50	

Carbonate Content – HCl Reaction

The test method considers classes of carbonate content determined by visual observations according to 'Carbonate Content - Classification of Results', above. No single numerical value applies.

The visual observations are made after application of droplets of dilute hydrochloric acid (HCl) to a dried or a natural soil specimen. The term 'highly calcareous' is used where the method cannot distinguish between 'highly calcareous' and 'very highly calcareous or carbonate'.

Test references: BS 5930:2015+A1:2020, ISO 14688-1:2017

Carbonate Content – Gas Volume

The carbonate content is determined by drying selected soil material to a constant mass in a 110 °C drying oven and measuring the volume of dissipated carbon dioxide (CO_2) upon reaction of the soil with hydrochloric acid (HCl). The carbonate

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content is calculated from calibration values and expressed as a percentage of dry mass of the original soil.

Test references: ISO 10693:1995, ISO 19901-8:2014

Carbonate Content – Rapid Titration

The carbonate content is determined by using a dried or a natural soil specimen and measuring the pressure of dissipated carbon dioxide (CO_2) upon reaction of the soil with hydrochloric acid (HCl). The amount of excess acid is determined by titration against sodium hydroxide. The carbonate content is calculated from the mass of the specimen, the concentration of hydrochloric acid and the volume of sodium hydroxide used. Carbonate content is expressed as a percentage of dry mass of the original soil.

Test reference: BS 1377-3:2018, ISO 19901-8:2014

Carbonate Content – Gas Pressure

The carbonate content is determined by using a dried or a natural soil specimen and measuring the pressure of dissipated carbon dioxide (CO_2) upon reaction of the soil with hydrochloric acid (HCl). The carbonate content is calculated from the mass of the specimen and the pressure increase after reaction by comparison with calibration values. For a natural soil, a correction factor is applied to correct for water content. Carbonate content is expressed as a percentage of dry mass of the original soil.

Test references: ASTM D4373-21, ISO 19901-8:2014

Carbon Content – Elementary Analysis

Total carbon (TC) content is determined by dry combustion of an air-dried soil specimen. The specimen is heated at high temperature (at least 900 °C) in a continuous flow of air so that any (organic and inorganic) carbon present in the soil is oxidised to carbon dioxide (CO_2). The gas is then allowed to cool and analysed by an infra-red detector.

For the determination of the total organic carbon (TOC) content, soil samples are treated (prior to TC analysis) with phosphoric acid to expel any inorganic carbonates.

Alternatively, TOC content may be calculated by determining TC and subtracting the carbon present as carbonate which can be determined using one of the methods listed under 'Carbonate Content'.

Carbon content (TC and TOC) is expressed as a percentage of dry mass of the original soil.

Test reference: ISO 10694:1995

Sulphate Content

The acid-soluble sulphate content of a soil sample is determined by extracting the sulphate in dilute hydrochloric acid and removing the insoluble residue by filtration. The filtrate is made up to volume and the concentration of sulphate in the filtrate is determined by Inductively Coupled Plasma Optical Emission Spectroscopy (ICP-OES). This instrument works by injecting the liquid extract of the sample into a stream of plasma at a temperature around 4 000 °C. This causes ionisation of all components of the extract. The ions produced are then analysed within the same instrument by optical emission spectrometry, comparing the emission levels of the sample with a standard calibration graph.

Acid-soluble sulphate content of soil is reported as % SO_4.

The water-soluble sulphate content of a soil sample is determined on a test portion that has been sieved and crushed through a 2 mm sieve and oven dried to 110 °C. The test portion is mixed with distilled water to prepare a 2:1 water:soil extract.

The concentration of sulphates in the filtrate is also determined by ICP-OES.

If no other anions are present, the water-soluble sulphate content can be expressed either in terms of the concentration of sulphates in the extract in mg/L or in terms of percentage of water-soluble sulphate (SO_4) in the soil sample.

If water-soluble sulphate content is provided in mg/L SO_3 one can convert this to mg/L SO_4 by multiplying SO_3 by a factor 1.2. For extractions other than 2:1, the multiplying factor is different.

If water-soluble sulphates are reported as mg/L SO_3 (or SO_4) (i.e. the sulphates content of the extract) and a percentage is required, the reported value should be divided by 5 000 to obtain the sulphate content (in %) of the soil, and by 10 000 to obtain the sulphate content (in %) of the 2:1 water:soil extract.

Test reference: BS 1377-3:2018

Chloride Content

The acid-soluble chloride content of a soil sample is determined by dissolving the chloride in the sample in nitric acid and removing the insoluble material by filtration. The dissolved chloride is analysed by Volhard's method. The chloride in solution is precipitated with a known excess of standard silver nitrate. The excess silver nitrate is titrated against standard ammonium thiocyanate using ferric alum as an indicator. The colour change is white to orange/red.

The water-soluble chloride content of a soil sample is determined on a test portion that has been sieved and crushed through a 2 mm sieve and oven dried to 110 °C. The test portion is mixed with distilled water to prepare a 2:1 water:soil extract.

In the Mohr's method, a chloride ion will precipitate with silver nitrate. The chloride reacts with the silver ion before any silver chromate forms, due to the lower solubility of silver nitrate. The potassium chromate indicator reacts with excess silver ion to form a red silver chromate precipitate. The end point is the appearance of the first permanent orange colour. The chloride content is expressed as a percentage by mass of dry soil.

This test method is suitable for analysing solutions with a pH between 6.0 and 8.5.

Test references: in house, BS 1377-3:2018

Thermal Testing

Thermal Conductivity – Needle Probe

Thermal conductivity is defined as the amount of heat that passes through a unit cross-sectional area of a substance, under a unit thermal gradient, per unit time. The test specimen is allowed to reach temperature equilibration. A stainless steel thermal needle probe is then inserted into the specimen, ensuring that the probe heater is sufficiently buried in the soil and that a good probe/soil contact is made. After temperature equilibration is confirmed, a current is passed through the heater element in the probe. As a result, the temperature of the probe and the surrounding soil increases.

The thermal needle probe is calibrated using a block of material of known thermal conductivity.

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Thermal conductivity of soil is calculated from a suitable part of a temperature versus time curve (typically the last 2/3 of the values), using the following equation (Brandon & Mitchell, 1989):

$$k = \frac{P}{4\pi(T_2 - T_1)} \ln\left(\frac{t_2}{t_1}\right)$$

where:

k	=	Thermal conductivity [W/(m·K)]
P	=	Applied power [W/m]
T_2-T_1	=	Temperature change over selected time interval [K]
t_1, t_2	=	Start and end of time interval [s]

Test reference: ASTM D5334-22, ISO 19901-8:2014

Volumetric Heat Capacity – Needle Probe

Heat capacity is defined as the amount of heat to be supplied to an object to produce a unit change in its temperature. The SI unit of heat capacity is joule per kelvin (J/K). Volumetric heat capacity C is the heat capacity per unit of volume (J/m^3·K).

The test specimen is allowed to reach temperature equilibration. A stainless steel dual thermal needle probe is then inserted into the specimen, ensuring that the probes are sufficiently buried in the soil and that a good probe/soil contact is made. After temperature equilibration is confirmed, heat is applied to the heated needle for a set heating time while temperature is measured in the monitoring needle (distance between needles: 6 mm) during heating and during a cooling period following heating. Temperature measurements are taken at 1 s intervals.

From analysis of the temperature versus time relationship for the separated probes, thermal conductivity (k) and thermal diffusivity (D) are derived using specific algorithms, volumetric heat capacity (C) is subsequently calculated using the following equation:

$$C = \frac{k}{D}$$

where:

C	=	Volumetric heat capacity [MJ/m^3·K]
k	=	Thermal conductivity [W/(m·K)]
D	=	Thermal diffusivity [mm^2/s]

Test reference: Bristow et al. (1994)

Thermal Conductivity and Heat Capacity – Hot Disc

Thermal conductivity and volumetric heat capacity can be determined using a transient plane source (TPS) test using Hot Disk® equipment. A hot disc sensor consists of an electrically conducting pattern in the shape of a double spiral, which has been etched out of a thin metal foil. This spiral is sandwiched between two thin sheets of an insulating material (e.g. Kapton).

When performing TPS measurements, a plane hot disc sensor is fitted between the specimen and a disc of insulating material, or between two halves of a specimen. An electrical current is applied that is strong enough to increase the temperature of the sensor between a fraction of a degree up to several degrees. At the same time, the resistance (temperature) increase is recorded as a function of time. The hot disc sensor is used both as a heat source and as a dynamic temperature sensor.

Test reference: ISO 22007-2:2022

Electrical Resistivity Testing

Electrical Resistivity – Disc Electrode

Electrical resistivity is a measure of the resistance of soil to the flow of electric current. The test is performed using the disc electrode method. The metal disc electrodes are placed into contact with each end of a soil specimen, after which a series of direct current voltages is applied. The resulting current flowing between the electrodes is measured. The applied voltages are typically from 5 V to 10 V in 0.5 V steps, although lower voltages can be required in some cases.

Current readings are plotted against voltage to confirm that a linear relationship. If this is the case, the slope of the plot is calculated to give the resistance, R. The soil's resistivity is calculated using the following equation:

$$\rho = \frac{R \cdot A}{1000L}$$

where:

ρ	=	Soil electrical resistivity [Ω.m]
R	=	Measured resistance [Ω.s]
A	=	Cross-sectional area of specimen [mm^2]
L	=	Length of specimen [mm]

Test reference: BS 1377-3:2018

Permeability Testing

Constant Head Permeability – Triaxial Cell

The effect of stress level on the coefficient of permeability may be estimated from constant head tests in a triaxial cell – flexible wall permeameter. The specimen is compacted in a split mould by tamping of thin layers of moist soil to the required initial density, and subsequently mounted in the triaxial cell. Filter screens or porous disks are placed at both ends of the specimen. The required stress level is applied, and saturation is achieved by flushing with CO_2 as followed by controlled flow of de-aired water and the application of backpressure. The degree of saturation is checked by the pore water pressure response to small variations in cell pressure. A hydraulic gradient is applied, and the rate of flow is recorded for various time steps. The permeability is calculated in accordance with Darcy's equation for laminar flow.

Test references: ASTM D5084-16a, ISO 19901-8:2014, ISO 17892-11:2019

Constant Head Permeability – Permeameter

Coefficient of permeability can be derived from constant head testing in a rigid container, permeameter cell. A specimen is compacted in the permeameter by tamping of thin layers of moist soil to a required initial density. Filter screens or porous disks are placed at both ends of the specimen. Initial flow of water through the specimen is achieved by percolation of de-aired water from the bottom upwards, after which the permeameter cell is closed off. An inlet reservoir of de-aired water connects to the top of the permeameter cell and a hydraulic gradient is applied by adjustment of the inlet reservoir height in relation to the permeameter cell. Once a stable head condition is observed, the rate of flow is recorded, and the flow-phase of the test repeated for consistency checks. Permeability is calculated in accordance with Darcy's equation for laminar flow.

Test references: ASTM D2434-22, ISO 17892-11:2019, ISO 19901-8:2014

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One-dimensional Consolidation Testing

Oedometer – Incremental Loading

The oedometer test covers determination of the rate and magnitude of consolidation of a laterally restrained soil specimen, which is axially loaded in increments of constant stress until the excess pore water pressures have dissipated for each increment. Normally, each load increment is maintained for 24 hours.

The test is generally carried out on undisturbed (intact) cohesive specimens using a consolidometer (oedometer) apparatus, which is placed in a thermostatically controlled room (10 °C). Selection of mounting method depends on soil characteristics. Soils that show a tendency to swell, such as peat or overconsolidated clays, are mounted dry. Moist sponges are placed in the oedometer cell to retain sample moisture conditions. Other samples are usually mounted using the wet mounting method. Distilled water is added to the cell when loads are applied to the loading arm. When required, the initial load is increased to prevent swell.

Key parameters that can be obtained from this test are the preconsolidation pressure σ'_p and the coefficient of consolidation c_v. The preconsolidation pressure is estimated using the graphical Casagrande construction. The root time method or the log time method is used for determination of c_v. Other parameters that may be derived from this test are the compression index C_c, the coefficient of volume compressibility m_v and the vertical permeability k_v.

Test references: ASTM D2435/D2435M-11(2020), ISO 19901-8:2014, ISO 17892-5:2017

Oedometer – Constant Rate of Strain

The Constant Rate of Strain (CRS) oedometer test covers determination of the rate and magnitude of consolidation of a laterally restrained soil specimen when it is drained axially and subjected to controlled deformation loading. The rate of deformation is selected so that excess pore water pressures are between 3 % and 20 % of the applied axial stress. Drainage of pore water is permitted from the top of the specimen and pore water pressures are measured at the bottom of the specimen. The test is generally carried out on undisturbed (intact) cohesive specimens using a consolidometer, in a thermostatically controlled room (20 °C).

Key parameters that can be obtained from this test are the preconsolidation pressure σ'_p and the coefficient of consolidation c_v as a function of axial stress. The preconsolidation pressure is estimated using the graphical Casagrande procedure, while the coefficient of consolidation is determined analytically from the values of axial stress, strain and excess pore water pressure. Other parameters that may be derived from this test are the compression index C_c, the coefficient of volume compressibility m_v, the constrained modulus (M), and the coefficient of vertical permeability k_v.

Test references: ASTM D4186/D4186M-20e1, ISO 19901-8:2014

Strength Index Testing

Torvane and Pocket Penetrometer

The torvane and pocket penetrometer are small hand-held instruments for rapid index testing for determining undrained shear strength of undisturbed (s_u) or remoulded ($s_{u;r}$) specimens of fine (cohesive) soil. Multiple tests can allow calculation of sensitivity $S_t = s_u/s_{u;r}$.

The torvane test is carried out by pressing a standard vane into the soil and measuring the minimum torque required to rotate the vane. The vane size can be selected to suit the expected torque up to an equivalent undrained shear strength of the soil of 250 kPa. The undrained shear strength is correlated to the measured torque by vane size and torvane spring constant.

The pocket penetrometer test consists of pressing a small solid cylinder into the soil, to a specified penetration. The maximum force required for penetration is correlated to the undrained shear strength. The size of the cylinder can be selected so that undrained shear strength values of up to 900 kPa can be obtained.

Test references: ASTM D8121/D8121M-19, ISO 19901-8:2014

Needle Penetrometer

The needle penetrometer is a small held-held instrument for rapid strength index testing of cemented soil and soft rock. The test consists of pressing a needle into a laboratory specimen or in situ outcrop. The needle is a thin truncated cone with a minimum diameter of 0.3 mm, a maximum diameter of 0.8 mm and a cone angle of 20°. The maximum penetration is 10 mm. Force and penetration are recorded. Results are expressed as $NPR = F/D$, where F is the axial force in N and D is the penetration in mm. The axial force is limited to 100 N. NPR can be correlated to uni-axial compressive strength σ_c. Ulusay and Erguler (2012) suggest $\sigma_c = 0.042\ NPR^{0.929}$, where σ_c is in MPa and NPR is in N/mm.

Test references: ISRM (2014), Ulusay & Erguler (2012)

Fall Cone

The fall cone is a rapid index test for determining undrained shear strength of undisturbed (s_u) or remoulded ($s_{u;r}$) specimens of cohesive soil. Multiple tests can allow calculation of sensitivity $S_t = s_u/s_{u;r}$.

The test consists of suspending a standard cone of a specified mass and apex angle vertically over and just touching the surface of the specimen. Subsequently, the cone is released and penetrates the specimen under its self-weight. The depth of penetration for the selected cone is correlated to the undrained shear strength of the soil. Several correlations exist. The cone size and shape can be selected to suit the expected undrained shear strength of the specimen.

Test references: ISO 17892-6:2017, ISO 19901-8:2014

Hand Vane

The hand vane allows index testing for undrained shear strength of cohesive soil. The tool is similar to the laboratory miniature vane except for reduced control: manual penetration and rotation of the vane.

Several different types of undrained shear strength are possible:

a. Intact: undisturbed undrained shear strength (s_u) as derived using on an intact specimen;
b. Intact–residual: derived from post-peak data during initial shearing of an intact specimen;.
c. Intact–vane-remoulded: derived after multiple rotations of the hand vane after completion of the intact test;
d. Hand-remoulded: derived from steady state (post-peak if exists) resistance of a hand-remoulded test specimen;
e. Hand-remoulded–vane-remoulded: derived from steady state resistance of a hand-remoulded specimen, after applying multiple vane rotations.

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Different values of undrained shear strength of remoulded soil ($s_{u;r}$) are obtained from types b. to e. Multiple tests can allow calculation of sensitivity $S_t = s_u/s_{u;r}$.

A specimen may be tested in the sample tube in which it was taken, in a block sample or in a mould after removal from a sampler. The test apparatus consists of a rectangular vane with a short push rod for penetration into the soil. The vane is then slowly rotated by hand and the maximum torsional moment is recorded. Various vane sizes can be selected depending on the consistency of the specimen.

Calculation of undrained shear strength is based on a cylindrical failure surface for which uniform stress distributions are assumed. The equation for undrained shear strength is as follows:

$$s_u = \frac{T_{\max}}{\pi D^2 \left(\frac{1}{2}H + \frac{1}{6}D\right)}$$

where:

s_u = (peak) undrained shear strength [kPa]
T_{max} = maximum torsional moment [kNm]
D = vane diameter [m]
H = vane height [m]

Test reference: in-house

Laboratory Miniature Vane

The laboratory miniature vane test allows determination of undrained shear strength of cohesive soil. CEN (2007) classifies the laboratory miniature vane as a strength index test.

Several different types of undrained shear strength are possible:

a. Intact: undisturbed undrained shear strength (s_u) as derived using on an intact specimen;
b. Intact–residual: derived from post-peak data during initial shearing of an intact specimen;.
c. Intact–vane-remoulded: derived after multiple rotations of the hand vane after completion of the intact test;
d. Hand-remoulded: derived from steady state (post-peak if exists) resistance of a hand-remoulded test specimen;
e. Hand-remoulded–vane-remoulded: derived from steady state resistance of a hand-remoulded specimen, after applying multiple vane rotations.

Different values of undrained shear strength of remoulded soil ($s_{u;r}$) are obtained from types b. to e. Multiple tests can allow calculation of sensitivity $S_t = s_u/s_{u;r}$.

A specimen may be tested in the sample tube in which it was taken or in a mould after extrusion from the sample tube. The sample tube or mould is mounted in the test apparatus and a rectangular vane is lowered into the soil. The vane is then rotated at 10°/min (ISO 19901-8:2014) or at 60°/min to 90°/min (ASTM D4648/D4648M-16) and the maximum torsional moment is recorded. A continuous record of rotation versus torsional moment can also be made if required (optional). Various vane sizes can be selected depending on the consistency of the specimen. Calculation of undrained shear strength is based on a cylindrical failure surface for which uniform stress distributions are assumed. The equation for undrained shear strength is as follows:

$$s_u = \frac{T_{\max}}{\pi D^2 \left(\frac{1}{2}H + \frac{1}{6}D\right)}$$

where:

s_u = (peak) undrained shear strength [kPa]
T_{max} = maximum torsional moment [kNm]
D = vane diameter [m]
H = vane height [m]

Test references: ASTM D4648/D4648M-16, ISO 19901-8:2014

Unconsolidated Undrained Triaxial

This type of test is usually performed on undisturbed (intact) samples of cohesive soils. Depending on the consistency of the cohesive material, the test specimen is prepared by trimming the sample or by pushing a mould into the sample. A latex membrane with a thickness of approximately 0.2 mm is placed around the specimen. A lateral confining pressure of 600 kPa to 1 000 kPa is maintained during axial compression loading of the specimen. Some test procedures consider lateral confining pressures that are equivalent to total in situ vertical stress. Consolidation and drainage of pore water during testing is not allowed. The test is deformation controlled (strain rate of 60 %/h), single stage, and stopped when an axial strain of 15 % or 20 % is achieved. The deviator stress is calculated from the measured load assuming that the specimen deforms as a right cylinder.

The presentation of test results includes a plot of deviator stress versus axial strain. The undrained shear strength, s_u, is taken as half the maximum deviator stress. The stress at 15 % or 20 % strain is used to calculate s_u if a maximum stress has not been reached earlier. Derivation of s_u from a selected value of axial strain is also possible, but uncommon.

undrained shear strength of remoulded soil $s_{u;r}$ can be obtained by an additional test on a remoulded (compacted) specimen. When possible, the tested undisturbed specimen is kneaded in the membrane, and then reshaped in a mould prior to testing. Stiff to hard specimens are cut into pieces, and reconstituted (compacted) by tamping the pieces in layers into a mould, until the original specimen dimensions are obtained. Multiple tests can allow calculation of sensitivity $S_t = s_u/s_{u;r}$.

Test references: ASTM D2850-15, ISO 17892-8:2018, ISO 19901-8:2014

Sensitivity

Sensitivity is the ratio of undrained shear strength of undisturbed soil to undrained shear strength of remoulded soil, $S_t = s_u/s_{u;r}$.

Sensitivity can be derived from the results obtained by tests of the same type, performed for intact (undisturbed) and remoulded soil conditions. Typically, the material from the same soil sample or soil specimen is used, at the same soil unit weight γ. Note that values of S_t can depend on the selected test type. Values of S_t can be less than unity.

Test reference: ISO 19901-8:2014

Strength anisotropy ratio - undrained shear strength

Strength anisotropy ratio $s_{u;a1}/s_{u;a2}$ refers to directional dependence of a physical property, in this case undrained shear strength s_u. Many soils exhibit strength anisotropy.

Strength anisotropy ratios can be derived from results obtained by tests of the same type, performed in multiple directions relative to in situ conditions. A laboratory approach is common. An example would be to trim laboratory test specimens from a soil sample, such that tests can be conducted in two or more directions relative to the in situ orientation of the sample. Note that values of $s_{u;a1}/s_{u;a2}$ can depend on the selected test type.

An indication of strength anisotropy ratio can be obtained by comparing results of different test types, with different modes and/or different directions of shearing. An example of this approach would be to compare test results of consolidated undrained triaxial extension tests, direct simple shear tests, and

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consolidated undrained triaxial compression tests, while maintaining specimen orientation equal to the in situ orientation of the sample. Different test types introduce differences related to the test types, which can be difficult to distinguish from strength anisotropy.

Test reference: ISO 19901-8:2014

Strength and Stiffness Testing

Ring Shear

Ring shear tests are performed by deforming remoulded or reconstituted (compacted) specimen at a controlled displacement rate until a constant minimum drained shear resistance is reached on a shear plane, largely determined by the configuration of the apparatus. From this, the effective angle of internal friction at residual shear conditions (φ'_R) can be derived.

The ring shear apparatus enables an annular specimen of soil, 5 mm thick with internal and external diameters of 70 mm and 100 mm, respectively, to be subjected to (unlimited) rotational shear.

After completion of an initial consolidation stage, formation of a shear plane is induced by manually rotating the test assembly at a relatively fast rate (~450 mm/min is often used) for 1 to 2 full rotations. The specimen is then reconsolidated prior to a second stage of shearing. The second stage of shearing aims for drained conditions until ultimate residual shear resistance is achieved; this usually requires a displacement of ~20 mm. For the second stage, a rate of displacement 0.048 degrees/min is typically adequate for a large range of soils.

Optionally, two or more normal stresses can be applied to a single test specimen.

The presentation of the test results includes a plot of shear stress versus displacement at fast and slow shear stages where applicable.

Test references: ASTM D6467-21e1, ISO 19901-8:2014, ISO 17892-10:2018

Direct Simple Shear

Direct simple shear (DSS) tests provide a simulation of the plane strain mode of shearing for undisturbed (intact), remoulded or reconstituted (compacted) specimens. Key features of the DSS test are essentially constant horizontal dimensions of the specimen in the direction of shear, and a constant volume during shear to simulate undrained behaviour for a saturated test specimen. A constant volume is achieved by maintaining a constant specimen height. A constant specimen height is achieved by varying the normal load applied to the specimen or by fixing the vertical loading ram in place.

The direct simple shear test is carried out on a cylindrical specimen of 50 mm to 66 mm diameter and 16 mm to 30 mm height, depending on test apparatus. Lateral confinement of the specimen is provided by (1) a membrane in combination with a stack of brass shearing washers or by (2) a reinforced membrane. There are no facilities for applying back pressure and control of drainage.

The stress state within a test specimen is insufficiently uniform to allow fundamental processing of test results. Nevertheless, data are commonly presented by shear stresses and strains for the horizontal plane and by equivalent pore pressures.

The peak horizontal shear stress is inferred as the undrained shear strength.

Test reference: ASTM D6528-17, ISO 19901-8:2014

Direct Shear

Direct shear testing (or shear box testing) is a method for determining drained soil resistance (effective angle of internal friction, φ') for cohesionless and cohesive soils.

The soil to be tested is placed in a split mould, with internal dimensions of 60 mm by 60 mm. A porous stone and loading plate are placed on top of the specimen and a normal load is applied to the specimen. The sample is then sheared, by displacing the top half of the split mould relative to the bottom half, at a rate of displacement preventing significant excess pore-water pressures to be generated. During the test, horizontal displacement, load and vertical displacement are recorded.

On completion of the first stage, the specimen is removed from the mould and the unit weight and water content are determined. Two further tests may then be performed, at the same unit weight, but with increased normal loads.

The test results are presented in the form of graphs of horizontal displacement versus shear stress and normal stress versus maximum shear stress.

Test references: ISO 19901-8:2014, ISO 17892-10:2018

Consolidated Undrained Triaxial

The consolidated undrained triaxial test offers the opportunity to derive both undrained and drained strength parameters for undisturbed (intact) or remoulded (compacted) specimens. Specimens are generally prepared by trimming cohesive samples to the required dimensions. The wet mounting method is used, which includes use of wet porous disks and a water-filled drainage system.

Test procedures include specimen saturation, consolidation and compression loading. For cohesive soils, filter paper strips are attached to the specimen circumference to promote drainage during consolidation. Saturation is obtained by incrementing cell pressure and back pressure. The degree of saturation is checked by the pore water pressure response to small variations in cell pressure.

In case of isotropic consolidation (CIU) the specimen is usually consolidated to a stress level equivalent to the mean in situ stress estimated for the appropriate sample depth. For anisotropic consolidation (CAU), the specimen is consolidated to the estimated vertical and horizontal effective stresses. Various consolidation stages may be adopted to simulate the consolidation history and the effects of the expected loading sequence.

Specimen shearing is carried out under conditions of constant axial strain rate, while monitoring axial load and pore water pressure. A strain rate of 4 %/h is generally applied, except when consolidation was slow, in which case a smaller strain rate is applied. The deviator stress is calculated from the measured load assuming the specimen deforms as a right cylinder. The shearing stage is terminated based on effective principal stress ratio (ratio of effective axial stress to effective lateral stress σ'_1/σ'_3), or when an axial strain of 15 % or 20 % is reached. The CIU test may consist of three consolidation and shearing stages of increasing stress level. These stages may be performed on a single specimen or on three separate specimens.

The presentation of test results includes stress-strain data, effective stress paths, pore water pressures and shear strength parameters. Stress paths may be presented in terms of a mean

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effective stress (p' or s') and the principal stress difference or deviator stress (q or t) as follows:

- Cambridge p'-q space and ASTM p'-q space, with p' defined as $(\sigma'_1 + 2\sigma'_3)/3$ and q as $\sigma_1 - \sigma_3$;
- ISO 17892-9:2018 s'-t space, with s' defined as $(\sigma'_1 + \sigma'_3)/2$ and t as $(\sigma_1 - \sigma_3)/2$.

The undrained shear strength is defined as half the deviator stress at failure, $s_u = q/2$ and is typically reported for the following failure criteria:

1. maximum deviator stress q_{max};
2. maximum stress ratio q/p'_{max}.

The stress at 15 % or 20 % strain is used to calculate undrained shear strength when a maximum stress has not been reached. Alternative failure criteria include peak pore pressure u_{max}, limiting axial strain and (Skempton's) pore pressure parameter $\bar{A} = 0$.

A secant angle of internal friction, φ', is determined from $q = M \cdot p'$ where $M = (6sin\varphi')/(3\text{-}sin\varphi')$ or $sin\varphi' = 3M/(6 + M)$ for compression conditions. This definition assumes a zero effective cohesion intercept and may be applied to M_{max} but also to other values of M and corresponding values of q and p'. Similarly, $sin\varphi' = t/s'$. For tests with three shearing stages, angles of internal friction may be determined for each stage separately, and from a straight line approximation of the failure points of the three stages. The latter method also provides a value for effective cohesion intercept c'.

Test references: ASTM D4767-11(2020), ISO 19901-8:2014, ISO 17892-9:2018

Consolidated Drained Triaxial

Consolidated drained triaxial compression tests are generally performed on samples of cohesionless soils. The specimen of dry soil is prepared in the rubber membrane on the base of the triaxial cell, without the use of side drains. Soil particles larger than 20 % of the diameter of the specimen are removed. Specimens are prepared by tamping thin layers of soil to a density approximating the estimated in situ dry density. To saturate the specimen, CO_2 gas is used to expel the air and subsequently de-aired water is used to expel the CO_2 gas. The specimen is further saturated by incrementing cell pressure and back pressure, until the pore pressure response to a cell pressure increment (B-factor) indicates saturation is complete. The specimen is then isotropically or anisotropically consolidated (CID and CAD respectively).

After consolidation the sample is sheared by applying axial load at a sufficiently slow rate to permit drainage (usually 6 %/h). The lateral confining pressure is kept constant during each loading stage. Pore pressure measurements are made at the bottom to check if the test is fully drained. The deviator stress is calculated from the measured load assuming the specimen deforms as a right cylinder. The CID test may have three consolidation and loading stages of increasing pressure performed on either a single specimen or on three separate specimens. The CAD test is limited to a single shearing stage. A shearing stage is terminated on the basis of effective stress ratio (ratio of effective axial stress to effective lateral stress, σ'_1/σ'_3), or when an axial strain of 15 % or 20 % is reached.

Results include stress-strain data, stress paths, and volumetric/ shear strain of each loading stage. Stress paths may be presented in terms of a mean effective stress (p' or s') and the principal stress difference or deviator stress (q or t) as follows:

- Cambridge p'-q space and ASTM p'-q space, with p' defined as $(\sigma'_1 + 2\sigma'_3)/3$ and q as $\sigma_1 - \sigma_3$;
- ISO 17892-9:2018 s'-t space, with s' defined as $(\sigma'_1 + \sigma'_3)/2$ and t as $(\sigma_1 - \sigma_3)/2$.

A secant effective angle of internal friction, φ', is determined from $q = M \cdot p'$ where $M = (6sin\varphi')/(3\text{-}sin\varphi')$ or $sin\varphi' = 3M/(6 + M)$ for compression conditions. This definition assumes zero effective cohesion intercept and may be applied to M_{max} but also to other values of M and corresponding values of q and p'. Similarly, $sin\varphi' = t/s'$. For tests with three shearing stages, angles of internal friction may be determined for each stage separately, and from a straight line approximation of the failure points of the three stages. The latter method also provides a value for effective cohesion intercept c'.

Optionally, the attainment of critical state conditions may be assessed from filtering criteria based on Jeffries and Been (2016) as follows:

- Dilation rate, D: samples identified to reach a critical state by determining a mean dilation rate $D < 0.01$ towards the end of the test:

$$D = \frac{d\varepsilon_v^p}{d\varepsilon_q^p} \approx \frac{\delta\varepsilon_v}{\delta\varepsilon_q} \approx \frac{(\varepsilon_{v,i+1} - \varepsilon_{v,i})}{(\varepsilon_{q,i+1} - \varepsilon_{q,i})}$$

  where $\delta\varepsilon_v$ is change in total volumetric strain, $\delta\varepsilon_q$ is deviatoric strain across each increment (i to $i + 1$) used to derive D, ε_v is volumetric strain $\varepsilon_v = \varepsilon_a + 2\varepsilon_r$, $\varepsilon_q = 2/3(\varepsilon_a - \varepsilon_r)$ and subscripts a and r refer to axial and radial directions (compression positive);
- Relative density, D_r: test specimens with an initial estimated D_r < 75%, i.e. interpretation of critical state is not considered reliable for tests performed on initially dense to very dense test specimens, since significant shear banding may occur, causing localisation of stresses.

For a test specimen approaching critical state, an approximation of effective angle of internal friction at large strain (critical state, φ'_{cv}) may be derived from principal deviator and mean effective stress ratios at large strain.

Test references: ASTM D7181-20, ISO 19901-8:2014, ISO 17892-9:2018

Miniature T-bar Penetration Test and Miniature Ball Penetration Test

The miniature T-bar penetration test (MTPT) involves measurement of axial force representing resistance of soil to continuous penetration at a steady slow rate of a cylindrical rod (T-bar penetrometer) positioned perpendicular to the lower end of a push rod. The miniature ball penetration test (MBPT) is equivalent to the MTPT except that the T-bar is replaced by a sphere. Axial force is measured just above the T-bar or the ball. Some systems measure axial force at the top of the push rod. Common instrument characteristics are as follows:

- miniature T-bar penetrometer length of 75 mm and diameter of 12 mm;
- miniature ball penetrometer diameter of 34 mm;
- 11.3 mm push rod diameter;
- penetration rate of approximately 20 mm/s.

The applicability of the MTPT and MBPT is soft cohesive soil with an undrained shear strength s_u < 50 kPa. Both tests require a soil sample with a height 300 mm < h < 600 mm and a diameter of typically > 300 mm. The tests are conducted at atmospheric pressure and the sample is typically confined by a sampler (e.g. box corer) or by a sample liner.

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The test procedure consists of recording downward and upward penetration and extraction lengths and recording of penetration and extraction resistances (q_T or q_B) of the penetrometer. This is done from the surface of the sample to about 50 mm above the base of the sample. Extraction resistance near the top of the sample can be downward if a lump of soil adheres to the penetrometer. The definitions of q_T and q_B are as follows.

$$q_T \text{ or } q_B = Q_p/A_1 - (A_2/A_1)u_0 - (A_3/A_1) \cdot (\sigma_{vo} - u_0)$$

where:

q_T = T-bar penetration resistance.
q_B = ball penetration resistance.
A_1 = maximum cross-sectional area of the penetrometer perpendicular to the axis of the push rod.
A_2 = cross-sectional (steel) area at the push rod/ penetrometer connection.
A_3 = cross-sectional area of the push rod immediately above the penetrometer.
Q_p = measured axial force on the penetrometer, positive for downward push.
u_0 = hydrostatic pore pressure at penetration depth, relative to the top surface of the sample. This is a calculated value. Calculation takes no account of any water on top of the sample and any non-hydrostatic pore pressure conditions.
σ_{vo} = total vertical stress at the midpoint of T-bar or ball, relative to the top surface of the sample. This is a calculated value.

If required, penetration resistance $q_m = Q_p/A_1$ can be provided.

One or more cyclic penetration/ extraction phases can be implemented. This is optional. A cyclic phase typically consists of 10 cycles of upward and downward penetration with stroke length of at least 6 times the diameter of the penetrometer. A cyclic phase usually starts in the primary downward penetration phase.

MTPT and MBPT results allow derivation of undrained shear strength s_u. Derived values of undrained shear strength are obtained from $s_u = q_T/N_T$ *or* $s_u = q_B/N_B$, where N_T is a T-bar factor and N_B is a ball factor. Values for N_T and N_B are typically about 10 for clay, considering (1) the penetrometer to be completely surrounded by soil and (2) a reference laboratory strength, i.e. $s_{u;CAUc}$ undrained shear strength obtained by anisotropically consolidated undrained triaxial compression. Lower N_T and N_B factors can be considered for penetration depths at and close to the surface of the sample. Derivation of s_u is optional.

Remoulded undrained shear strength can be derived from $s_{u;r} = q_{Tn}/N_{T;r}$ where q_{Tn} is normally taken as (q_T for downward push – q_T for upward retraction)/2 at cycle n, usually the tenth cycle. Values for $N_{T;r}$ (and $N_{B;r}$) are in the order of 14 to 20, depending on the reference method for $s_{u;r}$. Determination of derived values for $s_{u;r}$ is optional.

Test references: In-house, ISO 19901-8:2014

Piezoceramic Bender Elements

Bender elements are small piezo-electric elements that can be mounted in the base pedestal and top cap of oedometer, triaxial or direct simple shear (DSS) apparatus. The bender element technique can be used at any stage of a test without interfering with the particular test itself. The preparation of the test specimen should be in accordance with the requirements for oedometer, triaxial and DSS tests.

The composition of the elements is such that the application of a waveform voltage across the plates causes a bending motion which, when the elements are installed in a soil specimen, propagates a shear wave through the soil. This behaviour is reversible to the effect that motion of the receiving element will generate a small waveform voltage. After electrical stimulation of the transmitter elements, the arrival time of the wave at the receiving element is derived using a high frequency oscilloscope. S-wave velocity v_s is calculated from the derived travel time and measured spacing between the bender elements. Values of small strain shear modulus (G_{max}) of the soil can be calculated from v_s using the relationship $G_{max} = \rho \cdot v_s^2$ where ρ is density of the soil.

Test references: ASTM D8295-19, ISO 19901-8:2014

Interface Testing

Ring Shear – Soil-steel Interface

Ring shear interface tests are performed with a remoulded or reconstituted (compacted) soil specimen and a steel surface with a selected surface roughness. The primary parameter derived from this test method is the angle of interface friction (δ) for the soil-steel interface.

The ring shear apparatus enables an annular specimen of soil, 5 mm thick with internal and external diameters of 70 mm and 100 mm, respectively, to be subjected to (unlimited) rotational shear.

The specimen is typically consolidated to a normal stress equivalent to a lateral stress condition relevant to pile design. After completion of the initial consolidation stage, the subsequent shearing phase consists of two stages. In the first stage, the specimen is subjected to a series of fast shearing 'pulses' for a total displacement of at least 1 m. The shearing pulses impose displacements of ~200 mm each at a rate of ~500 mm/min, separated by pause periods (at zero applied shear stress) of ~3 minutes. This stage is intended to simulate a displacement history of a soil element adjacent to a pile during driving. The specimen is then allowed to recover for a period of 24 h after which the specimen is reconsolidated to simulate a lateral stress condition after the pile has been installed. Reconsolidation is followed by a second shearing stage aimed at drained conditions until steady state conditions are reached (with a minimum displacement of ~20 mm). For the second stage, a rate of ~0.02 mm/min is typically required for plastic clays, using a Bromhead device.

The presentation of the test results includes a plot of shear stress ratio versus displacement, for the fast and slow shear stages where applicable.

Test references: Jardine et al. (2005) (Appendix A), ISO 19901-8:2014

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Geotechnical Parameter Values

Scope

This document presents a summary of interpretation methods for derived values of geotechnical parameters. The definition of derived value is according to CEN (2009), CEN (2010), and ISO (2021a): 'value of a geotechnical parameter obtained from test results by theory, correlation or empiricism'.

Figure 1 illustrates geotechnical parameter values in the context of structure design or structure (re)assessment (e.g. ISO, 2015), where GP refers to geotechnical parameter values, GM refers to ground model, and SC to wider site characterisation.

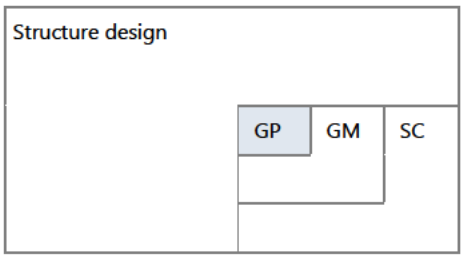

Figure 1: General context for geotechnical parameter values

The focus of this document is on:

- Mechanical properties of in situ soil, notably soil behaviour type and stress-strain-time dependent properties;
- Common data sources, i.e. ground intrusive technologies, particularly in situ testing, borehole geophysical logging, and laboratory testing;
- Methods that can provide parameter values by a transformation model and/or by empiricism, such as interpretation methods for cone penetration test (CPT) results.

This focus implies limited coverage of mechanical properties derived from (1) methods dedicated to obtaining derived values for a specific parameter, such as deriving undrained shear strength from an unconsolidated undrained triaxial compression test and (2) non-intrusive technologies (requiring no in-ground deployment of equipment), e.g. seismic reflection and seismic refraction.

Some of the presented methods suit computer-based interpretation of data records, machine learning and artificial intelligence.

The project-specific selection of methods and level of data integration depends on the agreed project requirements.

Procedure

Figure 2 outlines the procedure for development of geotechnical parameter values. Comments are given in 'Parameter Interpretation', below.

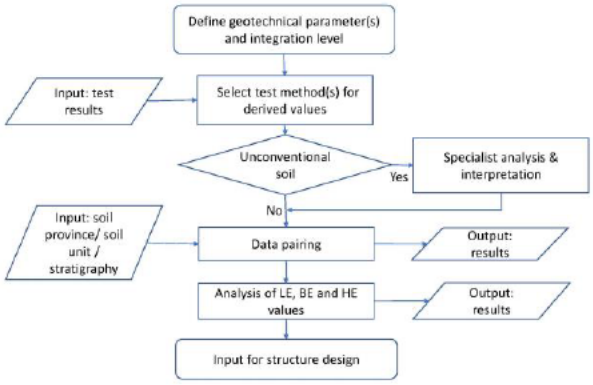

Figure 2: General procedure

Parameter Interpretation

Input for Structure Design

Derived values typically provide input for selection of characteristic values (e.g. CEN, 2009) and representative values (e.g. ISO, 2019) for geotechnical calculation models.

Conventional models are typically based on plasticity theory for ultimate limit states, and on elasticity theory and consolidation theory for serviceability limit states. Features of these geotechnical models are:

- Analysis of either drained (sand model) behaviour or undrained (clay model) behaviour for plasticity models;
- Analysis for the ultimate limit state differs from that for the serviceability limit state.

Many interpretation methods apply a transformation model to 'conventional' sands (drained soil behaviour) and clays (undrained soil behaviour). Drained or undrained behaviour (strain rate dependence) for the geotechnical analysis at hand may or may not coincide with respectively drained or undrained behaviour for a particular test method.

Integrated Geosciences

Project-specific integration levels for parameter interpretation can range from (1) a basic scope covering geodata processing and presentation of relevant, site-specific data and comments on any apparent data anomalies to (2) integrated use of multidisciplinary (e.g. geotechnical and geophysical) data that include representative ground profiles of derived values of geotechnical parameters.

This document excludes details of advanced methods for integrated use of multidisciplinary data.

Applicability of Methods

Parameter interpretation typically considers applicability of methods for data acquisition and for data analysis. If a method is applied outside of the intended applicability, then parameter values can be obtained that are uncertain, incorrect and/or anomalous. In practice, the following approaches are taken for parameter values outside of intended applicability:

- Present the parameter values and apply appropriate judgement when using the results;
- Remove data by intervention (manually or by algorithms), such as by data clipping;
- Provide (approximate) corrections to parameter values, e.g. by post-processing.

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Examples are given below.

A particular method may not be appropriate for silts, sand/clay/gravel mixtures, varved or layered soils, gassy soils, underconsolidated soils, peats, carbonate soils, cemented soils, and residual soils. These unconventional soils can require specialist analysis and interpretation. ISO (2021a) includes a list of unconventional soils and potential difficulties.

Most methods for data acquisition and parameter interpretation consider applicability in terms of lower and upper limits of stress, strain and temperature. Such limits are typically given or implied for a particular test method. Parameter values outside of these ranges can be subject to high uncertainty.

Results of a cone penetration test depend on a standardised penetration rate. Inevitably, the penetration rate will (shortly) differ from the standardised penetration rate at the start of a test and at the end of a test or planned interruption of penetration. In some cases, data are clipped to remove sections with a non-standard penetration rate.

A transformation model can include assumptions about soil (un)disturbance and ambient stress conditions. This can mean that laboratory test results for a soil sample taken immediately below ground surface or immediately below the bottom of the borehole may be outside of the applicability of a transformation model. Similarly, a short section of cone penetration data immediately below ground surface or immediately below the bottom of the borehole can cause a situation of no or limited applicability.

Data Pairing

Data pairing can be required for obtaining derived values for some of the geotechnical parameters. For example, correlation of CPT net cone resistance q_n and laboratory undrained shear strength s_u for clays requires:

- Derived values for q_n;
- Derived values for s_u;
- Pairing of q_n data points and laboratory s_u data points applicable to a nearby, but not the same, location in space;
- A correlation equation such as $N_{kt} = q_n/s_u$, where N_{kt} is a cone factor.

Considerations for data pairing typically include:

- Data checks, data detrending and data enhancement by integrated geosciences, see above;
- Applicability of methods, including undrained, partially drained or drained conditions;
- Uncertainties of the parameter values (source geodata) for data pairing;
- Representativeness of soil specimens and/or in situ test zone, e.g. bias may be introduced by selecting laboratory test specimens from the more cohesive, homogeneous parts of samples, particularly where soil conditions are not uniform;
- Geo-spatial proximity between data points selected for pairing, particularly with respect to soil homogeneity and soil correlation length;
- Decision rules and engineering judgement for screening of datasets for retention or removal of data;
- Use of specific weight factors for e.g. data assessed of higher of lower quality or importance.

Low Estimate, Best Estimate, High Estimate Values

The project requirements can include assessment of *low estimate* (LE), *best estimate* (BE) and *high estimate* (HE) values for derived values.

Analysis of derived values for LE, BE and HE values can be performed and enhanced by one or more of the following methods:

- Application of statistical methods,
- Data pairing, including geodata from multiple sources;
- Use of relevant prior information, including public domain information;
- Judgement and opinion.

This document excludes further details of possible methods for geotechnical analysis on this topic.

CPT-based Interpretation Methods

CPT interpretation methods are mostly based on empirical correlations with limited theoretical backing. Data integration with other, complementary investigation techniques (such as geological analysis, borehole/sample logging and laboratory testing) can improve confidence levels.

CPT-based interpretation techniques discussed below are subject to limitations such as:

- CPT measurements, including measurement uncertainty (Peuchen & Terwindt, 2014 and 2015) and effects resulting from deployment method, initial embedment of a cone penetrometer, penetration interruption and inevitable loss of saturation of a pore pressure measuring system;
- CPT interpretation techniques can be indirect, i.e. requiring estimates of various other parameters. This is consistent with an integrated geotechnical investigation approach. Inevitably, this approach also includes some redundancy of data;
- Empirical correlations can rely on data pairing, for example pairing of CPT net cone resistance at a point in space with laboratory undrained shear strength applicable to another, nearby spatial position. Data pairing uncertainty can be limited by applying judgement;
- Empirical correlations can use reference parameters such as the undrained shear strength determined from a laboratory single-stage isotropically consolidated undrained triaxial compression test on an undisturbed specimen obtained by means of push sampling techniques (e.g. Van der Wal et al., 2010). The reference parameter may not be appropriate for the selected geotechnical model, and adjustment may be necessary. Also, adjustment for test conditions may be necessary, for example in situ temperature versus laboratory temperature;
- The cone penetration test offers limited direct information on serviceability limit states (deformation), as the penetration process imposes large strains in the surrounding soil. In comparison to ultimate limit states, better complementary data will usually be required;
- The interpretations typically apply to conditions as encountered at the time of the geotechnical investigation. Geological, environmental, and construction/operational factors may alter as-found conditions.

Laboratory Test Methods

Laboratory test standards often specify procedures for obtaining derived values, particularly where it is possible to obtain a derived value by means of a conversion model or theory. Such derived values are thus part of the laboratory test report. An example is the unconsolidated undrained triaxial compression test. Normalised load and displacement data are the basic measured values. The measured values and the use of theory allow the calculation of a derived value of undrained shear

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strength by consideration of principal stress conditions and a theoretical deformation model.

A project-specific scope design for laboratory test results can be of a confirmatory nature. An example would be a limited scope that targets confirmation of CPT-based methods rather than detailed site-specific integration and correlation of data.

Non-intrusive Technologies

Results of non-intrusive technologies can serve as important input data for deriving geotechnical parameter values for semi-continuous profiles along a geophysical survey line (2D) and for voxel (3D) models (e.g. ISO, 2021b; Nauroy et al., 1998; Carpentier et al., 2021). Key features of this approach typically include (1) general input from an initial ground model, (2) a training data set of non-intrusive data (e.g. seismic reflection data) and ground-truthing data (e.g. CPTs), (3) machine learning and (4) correlation of geotechnical parameter values for spatial positions with non-intrusive data points.

The training phase typically includes validation of derived geotechnical parameter values (e.g. CPT cone resistance or shear modulus at small strain) at control locations. The validation results at the control locations allow comparison of accuracy of the derived values at non-intrusive data points with derived values of the ground truthing data.

CPT Penetration Behaviour

Soil behaviour during cone penetration testing shows large displacements in the immediate vicinity of the penetrometer, and small elastic displacements further away from the penetrometer. Density/structure, stiffness and in situ stress conditions significantly affect the measured parameters.

The measured cone resistance (q_c) includes hydrostatic water pressures as well as induced pore pressures resulting from stresses and strains related to the penetration process. The induced pore pressures are usually negligible for clean sand because the ratio of effective stress to pore pressure is high. This ratio can be low for penetration into normally consolidated and slightly overconsolidated clays. Knowledge of pore pressures around the penetrometer can thus be important. CPT parameters that take account of pore pressure effects include corrected cone resistance (q_t), net cone resistance (q_n) and pore pressure ratio (B_q). These parameters can be calculated if piezocone penetration test (PCPT or CPTU) data are available. The influence of pore pressures on sleeve friction f_s is relatively small. It is common to ignore this influence. Calculation of friction ratio R_f (defined as f_s/q_c) includes no allowance for pore pressure effects.

The penetration rate with respect to soil permeability determines whether soil behaviour is primarily undrained, drained, or partially drained. Partial drainage may also be denoted as partial consolidation. In general, soil behaviour during cone penetration testing is:

- Drained in clean sand, i.e. no measurable pore pressures because of (1) soil displacements and (2) soil volume change depending on dilative/contractive soil behaviour;
- Undrained in clay, i.e. no significant soil volume change immediately around the cone penetrometer and pore pressure change depending on dilative/contractive soil behaviour;
- Partially drained in soils with intermediate permeability, such as sandy silt, i.e. potential for (1) some soil volume change depending on dilative/contractive soil behaviour and (2) potential for pore pressure change depending on dilative/ contractive soil behaviour.

Results of a pore pressure dissipation test can provide indications for partial drainage conditions. Particularly, partial drainage conditions should be considered when t_{50} is less than about 100 s (DeJong & Randolph, 2012). The term t_{50} represents the time for 50 % dissipation of excess pore pressure at the u_2 location of a cone penetrometer.

CPT parameters can be influenced by the presence of thin (< 0.2 m thick) layers in a ground profile. Boulanger and DeJong (2018) proposed a method that provides estimates of corrected q_c and f_s values based on an inverse filtering procedure that accounts for thin layer and transitional effects during cone penetration.

The following sections mostly consider interpretation of drained soil behaviour (sand) and undrained soil behaviour (clay).

CPT-based Soil Behaviour Type Identification

Figures 3, 4 and 5 show soil behaviour type identification according to procedures given by Robertson (2009), representing an update of Robertson (1990) and Robertson (1991) respectively by exchange of Q_t with Q_{tn}. The procedures consider a normalised soil behaviour classification that provides general guidance on likely soil type (silty sand for example) and a preliminary indication of parameters such as angle of internal friction φ', overconsolidation ratio (OCR) and clay sensitivity (S_t). Classification is in general possible for $1 \leq Q_{tn} \leq 1000$, $0.1 \leq F_r \leq 10$ and $-0.5 \leq B_q \leq 1.4$, with exceptions for classification using a Q_{tn}-B_q chart for the areas where no classification is presented.

Classification charts use following soil behaviour type zones:

1. Sensitive, fine grained
2. Organic soils – peats
3. Clays – clay to silty clay
4. Silt mixtures – clayey silt to silty clay
5. Sand mixtures – silty sand to sandy silt
6. Sands – clean sand to silty sand
7. Gravelly sand to sand
8. Very stiff sand to clayey sand*
9. Very stiff, fine grained*

(*) Heavily overconsolidated or cemented

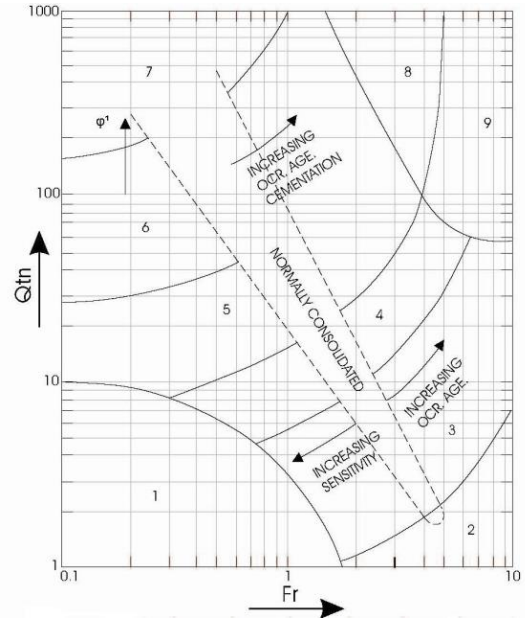

Figure 3: Classification chart Robertson (1990) with exchange of Q_t with Q_{tn}

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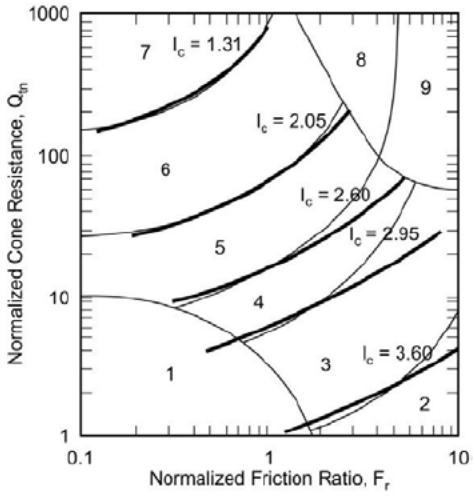

Figure 4: Soil behaviour type index I_c superimposed on Robertson (2009) classification chart

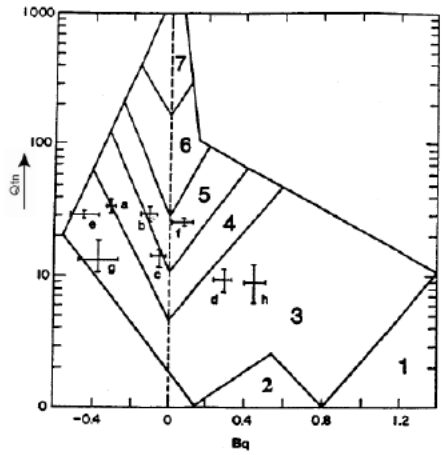

Figure 5: Classification chart Robertson (1991) with exchange of Q_t with Q_{tn}

The procedures require piezocone test data:

$$Q_{tn} = [(q_t - \sigma_{v0})/P_a](P_a/\sigma'_{v0})^n \qquad Q_t = \frac{q_t - \sigma_{vo}}{\sigma'_{vo}}$$

$$F_r \text{ or } nR_f = \frac{f_s}{q_t - \sigma_{vo}} 100\% \qquad B_q = \frac{u - u_0}{q_t - \sigma_{vo}}$$

where:

B_q = pore pressure ratio
F_r = normalised friction ratio
Q_{tn} = normalised cone resistance with variable stress exponent
Q_t = normalised cone resistance
q_t = corrected cone resistance
σ_{vo} = total in situ vertical stress
σ'_{vo} = effective in situ vertical stress
P_a = atmospheric pressure
n = stress exponent
f_s = measured sleeve friction
u = measured pore pressure
u_0 = theoretical hydrostatic pore pressure.

The stress exponent n is according to Zhang et al. (2002):

$$n = 0.381\ (I_c) + 0.05\ (\sigma'_{vo}/P_a) - 0.15$$

where $n \leq 1$.

Robertson and Wride (1998) defined soil behaviour type index I_c as follows:

$$I_c = [(3.47 - \log Q_{tn})^2 + (\log F_r + 1.22)^2]^{0.5}$$

Soils with $I_c < 2.05$ are generally cohesionless, coarse grained, where cone penetration is generally drained and soils with $I_c > 2.60$ are generally cohesive, fine grained, where cone penetration is generally undrained (Robertson & Wride, 1998). Cone penetration in soils with $2.05 < I_c < 2.60$ is often partially drained.

Figure 6 presents a classification chart for friction cone data according to Robertson (2010). This procedure requires no pore pressure input. A non-normalised soil behaviour type index, I_{SBT} applies:

$$I_{SBT} = [(3.47 - \log (q_c/P_a))^2 + (\log R_f + 1.22)^2]^{0.5}$$

I_{SBT} is similar to I_c. Values for I_{SBT} and I_c are typically comparable for effective in situ vertical stress between 50 kPa and 150 kPa.

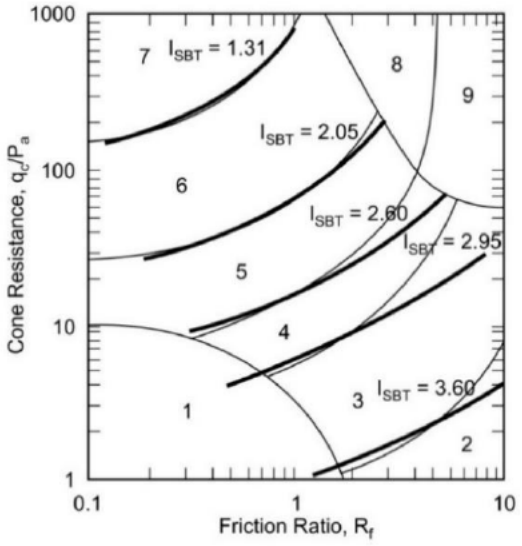

Figure 6: Robertson (2010) classification chart including I_{SBT}

Figure 7 presents a classification chart focusing on contractive and dilative soil behaviour, according to Robertson (2016a). The equations for the contractive-dilative boundary (CD) and soil behaviour type index (I_B) are as follows:

$$CD = (Q_{tn} - 11)(1 + 0.06F_r)^{17} \text{ and}$$

$$I_B = 100(Q_{tn} + 10)/(70 + Q_{tn}F_r)$$

Suggested values of CD are $CD = 60$ (low value) and $CD = 70$ (high value). Suggested values for I_B are $I_B = 32$, representing a low value for sand-like soil behaviour types and $I_B = 22$ representing a high value for clay-like soil behaviour types. The region between $I_B = 32$ and $I_B = 22$ represents soils typically showing transitional or intermediate soil behaviour types.

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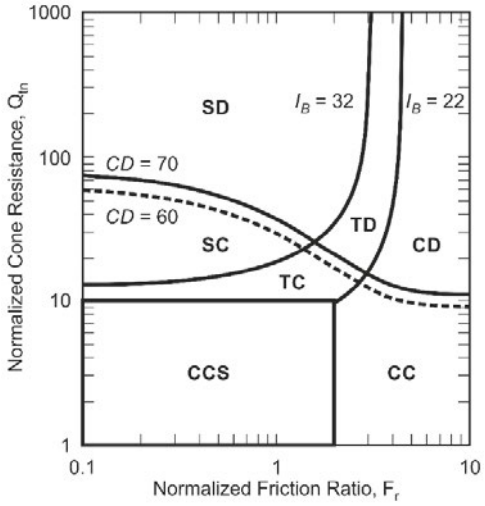

Figure 7: Classification chart according to Robertson (2016a)

CCS Clay-like – Contractive - Sensitive
CC Clay-like – Contractive
CD Clay-like – Dilative
TC Transitional – Contractive
TD Transitional – Dilative
SC Sand-like – Contractive
SD Sand-like – Dilative

Sand Model – CPT-based Methods

Unit Weight – Sand

Unit weight of uncemented (silica) sand, silt and clay soils may be derived according to Mayne et al. (2010):

$$\gamma = 1.95\gamma_w \left(\frac{\sigma'_{vo}}{P_a}\right)^{0.06} \left(\frac{f_t}{P_a}\right)^{0.06}$$

where total unit weight γ and unit weight of water γ_w are in kN/m^3 and effective in situ vertical stress σ'_{vo} is in kPa. The symbol f_t refers to sleeve friction corrected for pore pressures acting on the end areas of the friction sleeve, with units in kPa. Atmospheric pressure P_a is in kPa.

Unit weight may also be derived according to Lengkeek et al. (2018):

$$\gamma = \gamma_{ref} - \beta \cdot (\log(q_{t,ref}/q_t))/(\log(R_{f,ref}/R_f))$$

where γ_{ref} is a reference unit weight at which q_t is constant regardless of friction ratio R_f, β is a factor for unit weight contouring, $q_{t,ref}$ is a reference for total cone resistance q_t at which γ is constant regardless of R_f, and $R_{f,ref}$ is a reference friction ratio. The default values are: $\gamma_{ref} = 19$ kN/m^3, $\beta = 4.12$, $q_{t,ref} = 5$ MPa, and $R_{f,ref} = 30$ %. The correlation allows development of project-specific estimation of unit weight.

Shear Wave Velocity – Sand

If no in situ measurements of shear wave velocities (v_s) are available, then empirical correlation with CPT parameters may be considered. Hegazy and Mayne (2006) published a statistical correlation derived from 73 sites worldwide representing a range of soil types including sands, clays, soil mixtures and mine tailings (Figure 8). The correlation considers a normalized cone resistance (q_{c1N_hm}) and a soil behaviour type index (I_{c_hm}) as follows:

$$v_s = 0.0831 q_{c1N_hm} (\sigma'_{vo}/P_a)^{0.25} e^{(1.786 I_{c_hm})}$$

(Hegazy & Mayne, 2006)

where shear wave velocity v_s is in m/s and q_{c1N_hm} and I_{c_hm} are dimensionless. Calculations for q_{c1N_hm} and I_{c_hm} require iteration and consider cone resistance q_c or corrected cone resistance q_t, sleeve friction f_s, total in situ vertical stress σ_{vo}, effective in situ vertical stress σ'_{vo} and atmospheric pressure P_a.

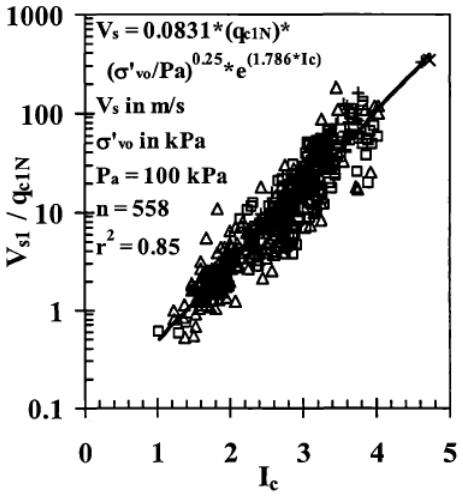

Figure 8: v_s – q_c correlation according to Hegazy & Mayne (2006)

Robertson and Cabal (2015) present a v_s correlation incorporating net cone resistance q_n ($= q_t - \sigma_{vo}$) and soil behaviour type index (I_c) as defined by Robertson and Wride (1998):

$$v_s = [\alpha_{vs}(q_t - \sigma_{vo})/P_a]^{0.5} \text{ where } \alpha_{vs} = 10^{(0.55\, I_c + 1.68)}$$

(Robertson & Cabal, 2015)

where shear wave velocity v_s is in m/s and corrected cone resistance q_t, total in situ vertical stress σ_{vo} and atmospheric pressure P_a are in kPa. The method can be applied to a wide range of soil behaviour types, notably uncemented Holocene to Pleistocene age soils. Older deposits could have a higher shear wave velocity. Exceptions are Zones 1, 8 and 9 of Robertson (1990 and 2009).

Baldi et al. (1989) derived a correlation between shear wave velocity v_s and cone resistance q_c for uncemented silica sands. This correlation is based on data from CPT, cross-hole, and Seismic cone penetration tests (SCPTs) performed in quaternary deposits of the predominantly silica Po River sand and Gioia Tauro sand with gravel.

$$v_s = 277 q_c{}^{0.13} \sigma'_{vo}{}^{0.27}$$

(Baldi et al., 1989)

where shear wave velocity v_s is in m/s and cone resistance q_c and effective in situ vertical stress σ'_{vo} are in MPa.

Shear wave velocity may be normalised according to Robertson and Cabal (2015):

$$v_{s1} = v_s \cdot (P_a/\sigma'_{vo})^{0.25}$$

(Robertson & Cabal, 2015)

In Situ Stress Conditions – Sand

A knowledge of in situ stress conditions is required for estimation of parameters such as relative density D_r and angle of internal friction of a sand deposit φ'. The effective in situ vertical stress σ'_{vo} may be calculated with a reasonable degree of accuracy but the effective in situ horizontal stress $\sigma'_{ho} = K_0 \cdot \sigma'_{vo}$ is generally unknown. Usually, it is necessary to consider a range of conditions for K_0 (coefficient of earth pressure at rest). The range can consider overconsolidation as inferred from a geological assessment, preconsolidation pressures of intermediate clay layers and/or theoretical limits of K_0.

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Geological factors concerning overconsolidation include ice loading, soil loading and groundwater fluctuations (influence from desiccation). Possible subdivisions for these factors are mechanical, suction, cyclic and ageing consolidation.

The following approach can be applied for direct estimation of K_0 based on Agaiby and Mayne (2019):

$$K_0 = 0.45\sqrt{(OCR)}$$

using: $OCR = \frac{\sigma'_p}{\sigma'_{v0}}$ $\sigma'_p = 0.33 \cdot q_n^{m'}$ $m' = 1 - \frac{0.28}{1+\left(\frac{I_c}{2.64}\right)^{25}}$

where OCR is overconsolidation ratio, σ'_p is effective preconsolidation stress, σ'_{v0} is effective in situ vertical stress, q_n is net cone resistance in kPa and I_c is soil behaviour type index. Typical values for m' are 0.72, 0.8 and 0.85 for clean sands, silty sands, and silts respectively.

The $K_0 - OCR$ relationship represents a schematisation of $K_0 = (1 - sin\varphi') \cdot OCR^{sin\varphi'}$ proposed by Mayne and Kulhawy (1982). Mayne and Kulhawy (1982) investigated mechanical overconsolidation of reconstituted laboratory specimens for over 170 different soils. For many soil types (e.g. Mayne, 2020), it can be shown that the $K_0 = 0.45\sqrt{(OCR)}$ equation provides similar statistics to the Mayne and Kulhawy correlation using φ' (effective angle of internal friction):

$$K_0 = (1 - sin\varphi') \cdot OCR^{sin\varphi'}$$

Figure 9 presents an approximate CPT-based correlation for K_0 according to Robertson (2016b). K_0 limits are typically set to 0.5 and 2. Linear interpolation is applied for the region between $K_0 = 0.5$ and $K_0 = 2$.

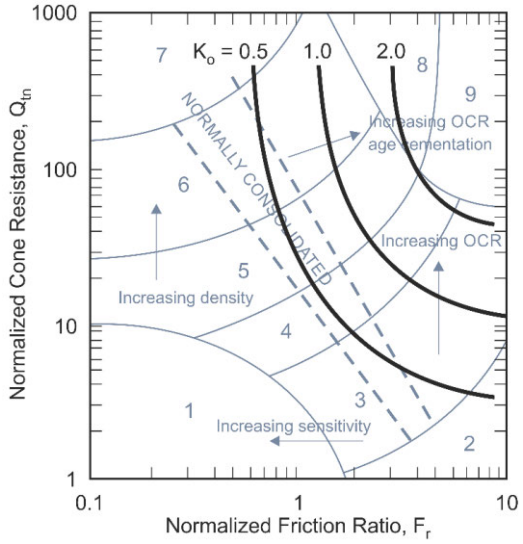

Figure 9: K_0 correlation according to Robertson (2016b)

No laboratory study can fully capture in situ behaviour. Particularly, K_0 may be underestimated if effects such as ageing and cyclic loading are relevant.

In general, in situ K_0 values are limited to the range $K_0 = 0.5$ to $K_0 = 1.5$. For many situations, K_0 values are believed to be relatively low at greater depths (say $K_0 < 1$ for depths exceeding 50 m). Jamiolkowski et al. (2003) recommend using a limiting value $K_0 = 1$ in practice, for limit states where low values of soil resistance and soil stiffness are critical.

Relative Density – Sand

The relative density concept applies to sands with a percentage fines of less than about 15 %.

Relative density is defined as $D_r = (e_{max} - e)/(e_{max} - e_{min})$, where e_{max} is maximum index void ratio, e represents in situ void ratio and e_{min} is minimum index void ratio. Maximum and minimum index void ratios are defined by laboratory testing. Relative density can exceed 100 %, because in situ void ratio can be lower than laboratory values for minimum index void ratio.

Determination of laboratory minimum and maximum index void ratios forms the basis for the relative density concept (loose, dense sand, etc.). No internationally agreed procedure is available.

CPT-based correlations are commonly used for estimation of in situ relative density. These correlations rely on database results of CPTs carried out in sand samples reconstituted in laboratory calibration chamber tests. Use of such correlations implies dependence on, for example:

- soil type of database versus soil type in situ;
- reference laboratory test method for determination of index void ratios, particularly sensitivity to minimum index void ratio;
- range of stress levels and K_0 values for calibration testing;
- results applicable to reconstituted sand samples, sample preparation method and soil stress history simplifications.

Calibration chamber test results apply to a limited range of stress conditions, typically:

50 kPa	<	σ'_{vo}	<	400 kPa
0.4	<	K_0	<	1.5

Sample preparation for laboratory chamber tests is usually by means of dry pluviation. Soil stress history application is by mechanical overconsolidation.

Jamiolkowski et al. (2003) proposed the following relationship between q_c and D_r for normally and overconsolidated silica (dry) sands:

$$D_{r(dry)} = \frac{1}{2.96}\ln\left[\frac{\frac{q_c}{P_a}}{24.94\left(\frac{\sigma'_{vo}\left(\frac{1+2K_o}{3}\right)}{P_a}\right)^{0.46}}\right]$$

and for saturated sands:

$$D_{r(sat)} = \left(\frac{-1.87 + 2.32\ln\frac{q_c}{(P_a * \sigma'_{vo})^{0.5}}}{100} + 1\right)\frac{D_r(dry)}{100}$$

where relative density D_r is a fraction. The correlation for saturated sands results in relative densities that can be up to about 10 % higher compared to the correlation for dry sands.

It is understood that Jamiolkowski et al. (2003) used results from one of the ASTM vibratory table methods for determination of minimum index void ratio. It is not clear which specific ASTM method was used, i.e. a vibratory table method requiring oven-dried soil or wet soil.

Kulhawy and Mayne (1990) proposed:

$$D_r^2 = Q_{tn*}/Q_f \qquad \text{where: } Q_{tn*} = \left(\frac{q_t}{P_a}\right)/\left(\frac{\sigma'_{v0}}{P_a}\right)^{0.5}$$

and q_t is corrected cone resistance, P_a is atmospheric pressure, σ'_{v0} is effective in situ vertical stress. Kulhawy and Mayne (1990) suggested using $Q_f = 280$ for highly compressible normally consolidated sands and $Q_f = 450$ for highly compressible overconsolidated sands (overconsolidation ratio of > 8) based on

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their dataset. Robertson and Cabal (2015) suggested $Q_f = 350$ for clean, uncemented, medium compressible quartz sands of about 1 000 years old. Values for Q_f can be closer to 300 for fine sands and closer to 400 for coarse sands. Furthermore, Q_f increases with age and increases significantly when age exceeds 10 000 years.

Krogh et al. (2021) consider an empirical correction for D_r estimates as obtained according to Jamiolkowski et al. (2003). The correction is for the upper few metres below ground surface ($\sigma'_{v0} < 50$ kPa). The equations are as follows:

$$D_{r(sat)} = \left(\frac{-1.87 + 2.32 \ln \frac{q_c}{(Pa * \sigma'_{vo})^{0.5}}}{100} + 1 \right) \frac{D_r(dry)}{100}$$

where $D_{r(dry)}$ is given by

$$D_{r(dry)} = \frac{1}{2.96} \ln \left[\frac{\frac{q_c}{P_a}}{24.94 \left(\frac{\sigma'_{vo}\left(\frac{1+2K_o}{3}\right)}{P_a} \right)^{0.46}} \right]$$

and where q_c is cone resistance, P_a is atmospheric pressure, σ'_{v0} is effective in situ vertical stress and K_0 is coefficient of earth pressure at rest.

Effective Angle of Internal Friction – Sand

The effective shear strength parameter φ' is not a true constant. It depends on factors such as density, stress level, shearing mode and mineralogy. There is evidence that overconsolidation ratio, method of deposition and in situ stress anisotropy is less important.

Correlation of angle of internal friction φ' to cone resistance q_c may be done at various levels of sophistication. Simple procedures rely on a conservative assessment of soil behaviour classification. A more sophisticated empirical correlation consists of:

a. Estimation of in situ stress conditions σ'_{vo} and σ'_{ho}
b. Estimation of relative density D_r
c. Empirical correlation of angle of internal friction φ' with D_r, σ'_{vo} and σ'_{ho}.

Estimation of stress conditions and relative density has been discussed above.

The empirical procedure proposed by Bolton (1986 and 1987) is used for estimation of φ'. This correlation applies to clean sands and considers peak secant angle of internal friction in Isotropically Consolidated Drained triaxial compression (CID) of reconstituted sand. This procedure requires estimation of the dilatancy index and the critical state angle of internal friction.

Kulhawy and Mayne (1990) determined an equation based upon 20 data sets obtained from calibration chamber tests. This equation is almost identical to the empirical formula determined earlier by Trofimenkov (1974) which was based on mechanical cone data. Mayne (2007) validated the use of total cone resistance q_t instead of cone resistance q_c used in the equation from Kulhawy and Mayne (1990).

$$\varphi' = 17.6 + 11.0 \log \left(\left(\frac{q_t}{P_a} \right) / \left(\frac{\sigma'_{vo}}{P_a} \right)^{0.5} \right) \qquad \text{(Mayne, 2007)}$$

Undrained Shear Strength – Sand

Kaltekis and Peuchen (2022) presented the following site-specific correlation between net cone resistance q_n and undrained shear strength s_u of sand and transitional soil (Figure 10):

$$\frac{s_u}{\sigma'_{v0}} = \begin{cases} 0.0096 \cdot \frac{q_n}{\sigma'_{v0}} - 0.4823 & \text{, for } \frac{q_n}{\sigma'_{v0}} \geq 124 \\ 0.703 & \text{, for } \frac{q_n}{\sigma'_{v0}} < 124 \end{cases}$$

where s_u, q_n and effective in situ vertical stress σ'_{v0} are in kPa.

The definition of s_u considers:

- Consolidated undrained triaxial test (ISO, 2018) as reference;
- Reconstituted specimens prepared by moist reconstitution and intact specimens;
- $(\sigma'_1/\sigma'_3)_{max}$ as failure criterion for undrained shear strength, where σ'_1 and σ'_3 are the effective principal stresses.

The applicability of this correlation to soils other than those of the source data (Southern North Sea) is unknown.

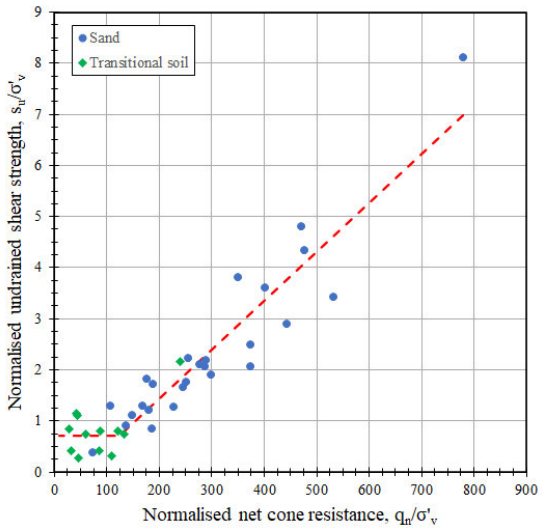

Figure 10: $s_u - q_n$ correlation according to Kaltekis and Peuchen (2022)

Constrained Modulus – Sand

Kulhawy and Mayne (1990) derived two formulas for the determination of the constrained modulus for both normally consolidated and overconsolidated sands by indicating that the modulus is a function of relative density. The determination of relative density can be done with, for example, the methods indicated above.

$$M = q_c * 10^{1.09 - 0.0075 D_r}$$
(normally consolidated sands, Kulhawy & Mayne, 1990)

$$M = q_c * 10^{1.78 - 0.0122 D_r}$$
(overconsolidated sands, Kulhawy & Mayne, 1990)

where D_r is in %, and q_c and M in kPa respectively.

Young's Modulus – Sand

A common guideline is an empirical correlation given by Baldi et al. (1989). The correlation is for silica-based sand and considers cone resistance q_c, in situ stress conditions and secant Young's modulus for drained stress change E'. The ratio of E'/q_c typically ranges from about 3 to 5 for recently deposited normally consolidated sands up to about $E'/q_c = 6$ to $E'/q_c = 25$ for overconsolidated sands. The correlation has been inferred from laboratory conditions; including CPTs in a calibration chamber and conventional triaxial compression tests on reconstituted sand samples. It takes account of the degree of deformation and overconsolidation. In this regard, it is noted that secant deformation moduli are strongly dependent on strain level: the

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elastic modulus increases with decreasing strain to an upper limit at about 10^{-4} % strain.

Shear Modulus at Small Strain – Sand

For estimation of initial (small strain) or dynamic shear moduli, ratios of G_{max}/q_c of between about 4 and 20 can be considered, in accordance with Baldi et al. (1989). The basis for this correlation is similar to that of secant Young's modulus, except that laboratory resonant column tests serve as reference instead of triaxial compression tests. Results of limited in situ seismic cross-hole and downhole tests provide an approximate check of this correlation.

Interpretation of small strain shear modulus can also be estimated from a correlation proposed by Rix and Stokoe (1991) in which data from calibration test measurements is compared to the correlation obtained between G_{max} and q_c by Baldi et al. (1989).

$$G_{max} = 1634(q_c)^{0.25}(\sigma'_{vo})^{0.375}$$

(Rix & Stokoe, 1991)

where G_{max}, q_c and σ'_{vo} are in kPa.

Clay Model – CPT-based Methods

Unit Weight – Clay

Empirical correlation between unit weight of clay and CPT parameters is as described in 'Unit Weight – Sand' above.

Shear Wave Velocity – Clay

Hegazy and Mayne (2006) and Roberson and Cabal (2015) present empirical correlations between shear wave velocity and CPT parameters for a wide range of soils including clays, as described in 'Shear Wave Velocity v_s – Sand' above. The Hegazy and Mayne correlation is sensitive to use of q_c or q_t. It should be used with caution for soils showing undrained or partially drained CPT response.

Mayne and Rix (1995) derived a correlation between shear wave velocity v_s and cone resistance q_c for intact and fissured clays. A database from Mayne and Rix (1993) was used including 31 different clay sites.

$$v_s = 1.75{q_c}^{0.627}$$

(Mayne & Rix, 1995)

where shear wave velocity v_s is in m/s and cone resistance q_c is in kPa.

In Situ Stress Conditions – Clay

Similar to sand, a knowledge of in situ stress conditions is generally necessary for estimation of other parameters such as consistency (soft, stiff, etc.) of a clay deposit and compressibility.

Calculation of the effective in situ vertical stress σ'_{vo} is reasonably accurate. A more approximate estimate applies to the effective in situ horizontal stress σ'_{ho}, or K_0 as $\sigma'_{ho} = K_0 \cdot \sigma'_{vo}$.

Direct correlations for interpretation of the coefficient of earth pressure at rest K_0 are the same as described for the Sand Model.

For normally consolidated clays and silts, K_{0nc} may be correlated with angle of internal friction, in accordance with Jaky (1944), or more simply, in accordance with Mayne and Kulhawy (1982):

$$K_0 = (1 - sin\varphi') \cdot OCR^{sin\varphi'}$$

where OCR is overconsolidation ratio and φ' is effective angle of internal friction. For many types of clay (e.g. Agaiby & Mayne, 2020), this equation can be approximated by $K_0 = 0.45\sqrt{(OCR)}$.

The plasticity index together with OCR may also be used for preliminary estimates of K_{0oc} as indicated by Brooker and Ireland (1965).

Overconsolidation Ratio – Clay

Overconsolidation ratio is defined as $OCR = \sigma'_p/\sigma'_{vo}$ where σ'_p is the effective preconsolidation stress considered to correspond with the maximum vertical effective stress to which the soil has been subjected in the past, and σ'_{vo} is the current effective in situ vertical stress. The effective preconsolidation stress approximates a stress level where relatively small strains are separated from relatively large strains occurring on the virgin compression stress range. The reference OCR is usually based on laboratory oedometer tests carried out on undisturbed samples. It may thus be influenced by factors such as sample disturbance, strain rate effects and interpretation procedure.

The following approach can be applied (Agaiby and Mayne, 2019):

$$OCR = \frac{\sigma'_p}{\sigma'_{v0}} \qquad \sigma'_p = 0.33 \cdot q_n^{m'} \qquad m' = 1 - \frac{0.28}{1+\left(\frac{I_c}{2.65}\right)^{25}}$$

where OCR is overconsolidation ratio, σ'_p is effective preconsolidation stress, σ'_{v0} is effective in situ vertical stress, q_n is net cone resistance in kPa and I_c is soil behaviour type index. Typical values for m' are $m' = 0.9$ for sensitive clays and organic clays, $m' = 1.0$ for intact clays and $m' = 1.1$ for fissured clays.

Chen and Mayne (1996) presented the following correlation for 205 clay sites around the world:

$$OCR = 0.317 \cdot Q_t$$

Overconsolidation ratio may also be inferred indirectly from a geological assessment and from undrained strength ratios. Geological factors concerning overconsolidation have been discussed under 'in situ stress conditions - sand'. An empirical procedure for estimation of OCR based on undrained strength ratio s_u/σ'_{vo} is given by Wroth (1984). The procedure uses the strength rebound parameter Λ. Guidance for selection of Λ and normally consolidated undrained strength ratio is given by Mayne (1988). Historically, much use has also been made of the Skempton (1957) relationship between normally consolidated undrained strength ratio and plasticity index I_P. This equation is useful for preliminary estimates, considering that I_P probably relates to ϕ' in some complex manner.

Undrained Shear Strength – Clay

No single undrained shear strength exists. The in situ undrained shear strength s_u depends on factors such as mode of failure, stress history, anisotropy, strain rate and temperature.

Various theoretical and empirical procedures are available to correlate q_c with s_u. Theoretical approaches use bearing capacity, cavity expansion or steady penetration solutions, all of which require several simplifying assumptions. Empirical approaches are more common in engineering practice because of difficulties in realistic soil modelling. An empirical correlation for soft to stiff, intact, and relatively homogeneous clays is given by Battaglio et al. (1986) as follows:

$$s_u = (q_c - \sigma_{vo})/N_c$$

where s_u, σ_{vo} and q_c are in kPa. N_c is an empirical factor that typically ranges between 10 and 25. The higher N_c factors typically apply to clays with a relatively low plasticity index and/or apply to heavily overconsolidated clays. Lower N_c factors are generally appropriate for normally consolidated and slightly overconsolidated clays. The reference undrained shear strength

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is that determined from in situ vane test results. The term σ_{vo} (total in situ vertical stress) becomes insignificant for stiff clays at shallow depth so that the equation reduces to $s_u = q_c/N_c$.

If piezocone test data are available, then improved correlations are feasible because of the pore pressure information. Empirical correlations of piezocone test results with laboratory undrained shear strengths are commonly expressed, as follows:

$$s_u = q_n/N_{kt}$$

N_{kt} ranges typically between 8 and 30 with the higher N_{kt} factors applying to heavily overconsolidated clays.

Mayne and Peuchen (2022) account for N_{kt} variation according to B_q:

$$N_{kt} = 10.5 - 4.6 \cdot ln\,(B_q + 0.1)$$

where $B_q > -0.1$. The equation is based on 497 paired CPT and laboratory test results from 70 clay sites, particularly anisotropically consolidated triaxial compressive strength. Factoring of N_{kt} can be applied by multiplying the calculated N_{kt} factor by, for example, 0.85 and 1.2.

Mayne et al. (2015) recommend a mean $N_{kt} = 12$ with a standard deviation of 2.8 for correlation with laboratory anisotropically consolidated triaxial compressive strength. The recommendations are based on a study of 51 onshore and offshore clays and apply to normally consolidated to slightly overconsolidated clays with q_n values of typically less than 8 MPa. Slightly higher N_{kt} values can be expected for average laboratory undrained shear strength, defined as the average of laboratory triaxial compression, simple shear and triaxial extension.

Clay Sensitivity

The sensitivity of a clay (S_t) is the ratio of undisturbed undrained shear strength to remoulded undrained shear strength. Sensitivity may be assessed from the CPT friction ratio R_f, in accordance with Schmertmann (1978):

$$S_t = N_s/R_f$$

where N_s is a correlation factor typically ranging between 5 and 10. The correlation is expected to be inaccurate for sensitive clays where uncertainty in very low values for sleeve friction may dominate results.

The reference S_t value is often taken to be that determined from undisturbed and remoulded laboratory unconsolidated undrained triaxial tests. This reference S_t value may differ from that determined from other tests, for example laboratory miniature vane tests. This is partly related to the definition of sensitivity. For vane tests, several measurements of undrained shear strength are possible:

- Intact (I) = undisturbed undrained shear strength as measured on an intact/undisturbed specimen;
- Intact-Residual (I-R) = measured post peak during initial shearing of the intact specimen;
- Intact-Vane Remoulded (I-VR) = measured after multiple-quick rotations of the vane after completion of the intact test;
- Hand Remoulded (HR) = steady state (post-peak if exists) resistance of hand remoulded test specimen;
- Hand Remoulded – Vane Remoulded (HR-VR) = steady state resistance of hand remoulded specimen measured after applying multiple-quick vane rotations.

Skempton and Northey (1952) present a correlation of sensitivity and laboratory liquidity index I_L. This correlation may allow a check on CPT-based interpretation of sensitivity.

Effective Shear Strength Parameters – Clay

Measurement of pore water pressures during penetration testing has led to development of interpretation procedures for estimation of effective stress parameters of cohesive soils. Background information may be found in Sandven (1990). Currently available procedures are evaluated to be 'experimental' and are yet not commonly adopted.

In general, CPT interpretation of effective shear strength parameters for clay and silt relies on soil behaviour-type classification.

It is noted that significant silt and sand fractions in a clay deposit will increase φ', while a significant clay fraction in silt will decrease φ'.

Masood and Mitchell (1993) provide an equation for the determination of φ' by combining sleeve friction with the Rankine earth-pressure theory. The equation is based on the following assumptions:

- Unit adhesion between soil and sleeve is negligible;
- Friction angle between soil and sleeve = $\varphi'/3$;
- Lateral earth pressure coefficient during penetration is equal to the Rankine coefficient of lateral earth pressure under passive conditions.

$$\frac{f_s}{\sigma'_{vo}} = \tan^2(45° + \frac{\varphi'}{2})\tan(\frac{\varphi'}{3})$$

(Masood & Mitchell, 1993)

Mayne (2001) proposed an approximation of the Masood and Mitchell equation, as follows:

$$\varphi' = 30.8\left[\log(\frac{f_s}{\sigma'_{vo}}) + 1.26\right]$$ (Mayne, 2001)

Mayne (2001) also proposed the following approximation of friction angle φ' based on pore pressure ratio B_q and the cone resistance number N_m (Senneset, Sandven and Janbu, 1989):

$$\varphi' = 29.5{B_q}^{0.121}(0.256 + 0.336B_q + \log N_m)$$

(Mayne, 2001)

where

$$N_m = \frac{q_t - \sigma_{vo}}{\sigma'_{vo} + a}$$

where the cone resistance number N_m is dimensionless, total cone resistance q_t, total in situ vertical stress σ_{vo} and effective in situ vertical stress σ'_{vo} are in kPa.

Senneset et al. (1989) use the attraction value a as a function of soil type. In general, the attraction value ranges from 5 to > 50 for both sands and clays and may be estimated directly from CPT results. The correlation is valid if the angle of plastification β is zero. In general, a plastification angle of zero applies to medium sands and silts, sensitive clays and highly compressible clays.

Constrained Modulus – Clay

Mitchell and Gardner (1976) present an approximate correlation of cone resistance with constrained modulus M (or coefficient of volume compressibility m_v, where $M = 1/m_v$). Typical ratios of M/q_c range between 1 and 8 for silts and clays. Refinements include q_c ranges and soil type (silt, clay, low plasticity, high plasticity, etc.). The correlation relies on the results of conventional laboratory oedometer tests carried out on undisturbed clay and silt samples.

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Kulhawy and Mayne (1990) correlated constrained modulus M with net cone resistance data. This relationship is based on data from 12 (clay) test sites, with constrained moduli up to 60 MPa. The published standard deviation is 6.7 MPa.

$$M = 8.25\ q_n \qquad \text{(Kulhawy \& Mayne, 1990)}$$

Young's Modulus – Clay

Young's modulus E_u can be derived as follows:

- Estimation of undrained shear strength s_u from CPT data, as outlined above;
- Estimation of secant Young's moduli for undrained stress change in general accordance with correlations based on s_u, as presented by Ladd et al. (1977).

Laboratory undrained triaxial tests carried out on undisturbed clay specimen form the basis for the E_u versus s_u correlations. Typical E_u/s_u ratios at a shear stress ratio of 0.3 range between about 300 and 900 for normally consolidated clays and $E_u/s_u = 100$ to $E_u/s_u = 300$ for heavily overconsolidated clay. Higher E_u/s_u ratios would apply to lower shear stress ratios, and vice versa.

Shear Modulus at Small Strain – Clay

Mayne and Rix (1993) determined a relationship between G_{max} and q_c by studying 481 data sets from 31 sites all over the world. G_{max} ranged between about 0.7 MPa and 800 MPa.

$$G_{max} = 2.78\ {q_c}^{1.335} \qquad \text{(Mayne \& Rix, 1993)}$$

where G_{max} and q_c are in kPa.

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Positioning Survey and Depth Measurement

Introduction

This document describes survey of horizontal and elevation/depth reference points for geophysical, geotechnical and/or environmental data acquisition in a marine environment.

National and international standards for geophysical, geotechnical and/or environmental data acquisition (e.g. ASTM, BSI, CEN and ISO) require such surveys. This document summarises common practice.

Procedure

The procedure for positioning survey and depth measurement depends on the agreed project specifications. For example, water level correction and subsea positioning may not be part of the activities agreed upon. Some or all of the following steps can apply:

- Definition of the type of survey and the target location;
- Selection of coordinate reference system (CRS) according to ISO (2019)
- Set-up and initial checks of the survey system and depth measurement system;
- Surface positioning survey of the reference point, i.e., the determination of grid coordinates according to the selected CRS;
- Subsea positioning survey, i.e., adjustment of the surface positioning results for underwater offset;
- Sub-seafloor positioning survey, i.e., estimation of the spatial position of a data point below seafloor;
- Measurement of the water depth;
- Estimation of depth below water level for a data point applicable to the water column;
- Estimation of depth below seafloor for a data point applicable to the seabed;
- Calculation of elevation of seafloor relative to a vertical datum, e.g., water level correction;
- Calculation of elevation of a data point relative to a vertical datum according to the selected CRS.

This document uses the terms seafloor and seabed. Seafloor is the underwater ground surface, i.e. the plane separating water and ground (soil, rock, made ground). The seabed is the ground below seafloor. A synonym for seabed is sub-seafloor (ISO, 2021).

IHO Classification for Horizontal Positioning and Water Depth

The International Hydrographic Organization (IHO, 2020) defines 5 orders of hydrographic survey:

- IHO Order 2: areas where a general description of the seafloor is considered adequate;
- IHO Order 1b: areas where under-keel clearance is not considered to be an issue for the type of surface shipping expected to transit the area;
- IHO Order 1a: areas where under keel clearance is less critical but features of concern to surface shipping may exist;
- IHO Order Special: areas where under-keel clearance is critical;
- IHO Order Exclusive: areas where there is strict minimum under-keel clearance and manoeuvrability criteria.

The IHO requirements for spatial positioning are defined by (1) water depth and (2) a horizontal positioning component, both expressed as function of acquired water depth data.

The IHO Special Order and Exclusive Order surveys are uncommon in geotechnical and/or environmental data acquisition. A Special Order system set-up may consist of: RTK DGPS; a multibeam echosounder; a motion compensator, and a conductivity temperature depth (CTD) probe. Subsea positioning is uncommon in limited water depths.

An IHO Order 1a and 1b survey system set-up may include: high-accuracy DGPS; long baseline (LBL) subsea positioning; a CTD probe with Digiquartz pressure sensor; a barometer; and a tide gauge.

IHO Order 2 surveys are common in geotechnical and/or environmental data acquisition. Such system set-ups could include: DGPS; ultra-short baseline (USBL) subsea positioning (IMCA, 2017); CTD probe; single beam echosounder or direct sounding by drill pipe; a motion compensator; and predicted tide correction.

These are examples of the simplest set-ups. Independent measurements are often made using a redundant system (IOGP, 2011; ISO, 2021). For example, surface position may be determined by two independent DGPS systems or direct sounding by drill pipe and echosounding.

Accuracy Classes for Horizontal Position

Table 1 presents accuracy classes for geographical (horizontal, XY) position of a geodata point estimated for or acquired by e.g. geophysical investigation or geotechnical investigation. The Maximum XY Uncertainty is defined per accuracy class as a maximum allowable value for expanded measurement uncertainty with a coverage factor $k = 2$. The uncertainty is relative to a selected CRS. Conversion of the horizontal position if a data point to another CRS can introduce additional uncertainty.

Table 1: Position class and maximum XY uncertainty (Peuchen et al., 2023)

Accuracy Class for Horizontal Position	Maximum XY Uncertainty [m]
XY1	0.5
XY2	1
XY3	5
XY4	10
XY5	>10

Estimation of uncertainty of horizontal position can be done according to a Type B evaluation of standard uncertainty as defined by ISO (2008). A Type B evaluation is obtained by scientific judgement based on all of the available information on the possible variability of input quantities.

Accuracy Classes for Depth below Seafloor

ISO (2014) provides depth accuracy classes, as shown in Table 2. These classes apply to depth below seafloor of a data point or measurement point acquired by borehole logging, in situ testing, and physical sampling and laboratory testing. ISO (2014) includes guidance on factors to consider for data point depth uncertainty. One of the factors is the position of a sample or test specimen within a sampler.

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Class Z4 applies as default, except for samplers with no fixed seafloor reference, where Z5 applies as default.

Note that ISO (2014) uses accuracy class and application class interchangeably. A definition is given for application class and not for accuracy class. Application classes are defined in terms of 'classification of equipment based on achievable level of accuracy'. This is interpreted to mean achievable under favourable conditions.

Also, ISO (2014) does not define 'uncertainty'. Possible interpretations include combined standard uncertainty and expanded measurement uncertainty with a coverage factor of, say, $k = 2$. The terms standard uncertainty, expanded measurement uncertainty and coverage factor are defined by ISO (2007).

Table 2: Depth Accuracy Classes for Data Point Measurements Relative to Seafloor

Depth Accuracy Class	Maximum Data Point Depth Uncertainty [m]
Z1	0.1
Z2	0.5
Z3	1.0
Z4	2.0
Z5	> 2.0

Guidance on Use of Results

General

This section Guidance on Use of Results considers 'uncertainty' as expanded measurement uncertainty with a coverage factor $k = 2$, except where noted otherwise.

Use of results should consider that relatively complex uncertainty budgets (uncertainty estimates) can apply. Uncertainty budgets are typically project-specific, for example:

- Horizontal positioning of a tool at seafloor implies use of multiple instruments contributing to the uncertainty budget, e.g. (1) DGPS antenna position and (2) offset between antenna and actual position of the tool at seafloor;
- Soft soils can introduce uncertainty in underwater vertical position. A water pressure measurement tool mounted on an underwater frame may sink into the soil, thus affecting the measurement;
- Insufficient acoustic contrast between water and soft can affect echosounder water depth measurements;
- An irregular or sloping seafloor may affect echosounder measurements. An echosounder determines the earliest arrival of acoustic waves within the beam area. The highest points within the beam are assumed to correlate with the seafloor position, and thus yield the 'water depth'.

Horizontal positioning

- DGPS - antenna position uncertainty typically in the order of 1 to 2 metres;
- High accuracy DGPS - antenna position uncertainty typically in the order of 0.2;.
- RTK DGPS – antenna position uncertainty typically in the order of centimetres;
- Gyro compass – uncertainty typically in the order of 0.5° to 1°;
- Directional inclinometer - uncertainty typically in the order of 1° to 2° (relative to vertical) and in the order of 5° to 10° (orientation relative to North).

DGPS uncertainty contributions include the geodetic network, vessel dynamics and antenna offset. Continuous logging on location allows some quantification of position uncertainty.

The acquisition and analysis of directional inclinometer data for sub-seafloor positioning can apply to a drill string or a cone penetration test. This practice is uncommon.

Subsea positioning

- LBL system: receiver position uncertainty typically in the order of 1 metre;
- USBL system: uncertainty of typically 0.5 m plus 1 % of distance between transducer and transceiver.

Uncertainty contributions include timing, ray bending, sound absorption, noise and offset.

Water depth measurement

- Direct sounding by drill pipe: uncertainty of typically about 1 m plus 0.5 % of measured mean water depth;
- Echosounder: uncertainty of typically about 0.3 m plus 1 % of measured mean water depth;
- Digiquartz probe: probe position uncertainty of typically about 0.2 m plus 0.1 % of measured mean water depth;
- Motion compensator: heave measurements have a typical uncertainty of 0.05 m, and roll and pitch an uncertainty of about 0.1°, relative to the mounting of the unit itself.

The pressure sensor estimates are corrected for atmospheric pressure. The echosounder estimate typically incorporates CTD sound velocity checks, motion compensation, and transducer draught, including vessel squat correction. Vessel squat is a vertical displacement of the hull as a vessel moves. It is determined by water depth and the vessel shape and size. The direct sounding estimate includes uncertainties related to tape measurement, heave, drill pipe length variation due to self-weight and temperature change, drill pipe bending and offset from vertical axis.

Tide correction

- Predicted tides: correction uncertainty typically in the order of 0.2 m to 1 m, depending on tidal range and meteorological circumstances;
- High accuracy DGPS: antenna position uncertainty typically in the order of 0.1 m;
- Tide gauge: correction uncertainty typically in the order of 0.1 m;
- RTK DGPS: antenna position uncertainty typically in the order of 0.1 m.

Depth below seafloor – marine soil investigation

Peuchen et al. (2005) present the following expression for depth uncertainty assessment for in situ testing, i.e. excluding considerations for sampling and laboratory testing:

$$\Delta z = \pm\sqrt{a^2 + (b * d)^2 + (c * z)^2}$$

where:

- a constant depth uncertainty, i.e. the sum of all uncertainties that do not vary with depth below seafloor in metres
- b uncertainty dependent on water depth, i.e. the sum of all uncertainties that are water depth dependent
- c uncertainty dependent on data point depth below seafloor, i.e. the sum of all uncertainties that are data point depth dependent

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d water depth in metres
z data point depth in metres below to seafloor
Δz data point depth uncertainty in metres (95 % confidence level)

Tables 3 to 5 present coefficients and accompanying premises.

Table 3: Coefficients for Data Point Uncertainty Assessment – In Situ Testing

Deployment System	Data Point Depth Uncertainty Δz		
	a	b	c
Vessel drilling – favourable	0.4 m	0.003	0.003
Vessel drilling – adverse	1.0 m	0.005	0.004
Non-drilling – favourable	0.2 m	0	0.01
Non-drilling – adverse	0.8 m	0	0.02
Note: resolution estimated at 50 % of uncertainty			

Table 4: Premise to Estimated Data Point Depth Uncertainty – In Situ Testing and Vessel Drilling Deployment

Characteristics	Marine Setting	
	Favourable	Adverse
Vessel - horizontal position	Variation within 5 m of target	Variation within 5 m of target
Vessel heave	1 m at 'hook' point	3 m at 'hook' point
Tidal variation	1.5 m, with correction for tidal variation by pressure sensor mounted on seabed frame	3 m, with correction for tidal variation by pressure sensor mounted on seabed frame
Seafloor	Firm and level	Very soft seabed soils or very rugged seafloor
Drill pipe checkpoint	Touchdown on seabed frame at borehole start	Touchdown on seabed frame at borehole start
Drill pipe bending	None	Minor
Borehole orientation	Vertical	Inclined at average 2° from vertical from sea level to test depth z

Table 5: Premise to Estimated Data Point Depth Uncertainty – In Situ Testing and Non-Drilling Deployment

Characteristics	Marine Setting	
	Favourable	Adverse
Vessel - horizontal position	Variation within 5 m of target	Variation within 5 m of target
Vessel heave	1 m at 'hook' point	3 m at 'hook' point
Tidal variation	1.5 m	3 m
Seafloor	Firm and level	Very soft seabed soils or very rugged seafloor
Orientation of Penetration	Vertical at start, with correction for measured inclination	Inclined at average 5° from vertical from seafloor to test depth z

Peuchen and Wemmenhove (2020) present a probabilistic approach to depth uncertainty assessment for in situ testing data points and for sample depths, including sample recovery considerations.

Note that definition of seafloor is difficult for extremely soft ground. Reaction equipment may penetrate unnoticed into a near-fluid zone of the seabed. Settlement may also continue during testing (Bouwmeester et al., 2009).

Seabed frame settlement is likely to be governed by the following factors:

1. Descent velocity and penetration into seabed, including possible erosion (scouring) caused by seabed frame descent and resulting water overpressures;
2. Non-centric loading during touchdown and testing;
3. Variable on-bottom weight of reaction equipment, because of drilling, sampling and testing activities and because of tensioning and hysteresis forces in a heave compensation system;
4. Consolidation of seabed soil.

Depth below seafloor – marine geophysical investigation

ISO (2021) includes some guidance on uncertainties for depth below seafloor for data acquired by marine geophysical investigation, such as seismic reflection survey.

Uncertainties can typically be reduced by ground truthing, i.e. comparison and depth correction on the basis of marine soil investigation data.

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Appendix F

Supplementary Information about Document

Contents Appendix F: Supplementary Information about Document

List of Documents

F.1	Quality Management Record
F.2	Sustainability
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F.1 Quality Management Record

Fugro Project Lead: 5.1.2.e
Senior Project Engineer

Report Review: 5.1.2.e
Senior Geotechnical Consultant

Report Approval: 5.1.2.e
Principal Geotechnical Engineer

Document Section		Prepared By	Checked By
Main Text		5.1.2.e	
A	Plates referenced in Main Text		
B	Geotechnical Logs		
C	Results of Cone Penetration Tests		
D	Geotechnical Laboratory Test Results		
E	Descriptions of Methods and Practices		
F	Supplementary Information About Document		

Project Team

Initials	Name	Role
5.1.2.e	5.1.2.e	Senior Project Engineer
5.1.2.e	5.1.2.e	Senior Project Engineer

F.2 Sustainability

Fugro is committed to conducting its business ethically and responsibly and contributing to sustainable development. This requires balancing short and long term interests of our stakeholders and integrating economic, social, and environmental considerations into decision making. Fugro's vision is to contribute to a safe and liveable world. With its solutions, Fugro delivers an essential contribution to sustainability, as Fugro's services enable clients to make sustainable use of the Earth and its resources and build assets.

Fugro is participant in the United Nations Global Compact https://www.unglobalcompact.org/.

F.3 Document Issue Record

Section	Page No.	Plate No.	Issue	Revision
Main Text	All	-	02	Editorial updates
Appendix B	-	B-1 to B-4	-	Removed CPT interpreted log (superseded by geotechnical log)
Appendix B	-	B-1 to B-4	02	Included geotechnical log to 100.7 m BSF
Appendix C	-	C-1 to C-38	02	Addition of in situ test results up to 100.7 m BSF
Appendix D.2	-	D.2-1 to D.2-6	02	Included laboratory classification test results
Appendix D.3	-	D.3-1 to D.3-30	02	Included sample photographs
Main Text	All	-	03	Editorial updates
Appendix A	-	A-6 to A-7	03	Editorial updates
Appendix B	-	B-1 to B-4	03	Updates in descriptions based on in-office laboratory classification test results and editorial updates
Appendix C	-	C-1 to C-38	03	Editorial updates
Appendix D.1	All	-	03	Updated practice for geotechnical laboratory tests
Appendix D.2	-	D.2.1-1 to D.2.1-6	03	Included in-office laboratory classification test results
Appendix D.2	-	D.2.2-1 to D.2.2-10	03	Included particle size distribution test results
Appendix D.4	-	D.4-1 to D.4-20	03	Included consolidated drained triaxial test results
Appendix D.5	-	D.5-1 to D.5-20	03	Included ring shear test results
Appendix D.6	-	D.6-1 to D.6-2	03	Included direct shear test results
Main Text	3	-	04	Added reference to Plate A-3 'Coordinates and Water Depth' in section 2.1 and editorial updates
Appendix A	-	A-6 to A-7	04	Editorial updates
Appendix B	All	All	04	Updates in descriptions and editorial updates
Appendix C	All	C-1 to C-28	04	Corrected water depths and editorial updates
Appendix D.1	All	-	04	Editorial updates, updated practice for sample handling
Appendix D.2	-	D.2.1-1 to D.2.1-6		Editorial updates
Appendix D.5	1	-	04	Addition of section D.5.2 'References'
Appendix E	-	-	04	Updated appendices to latest issue

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