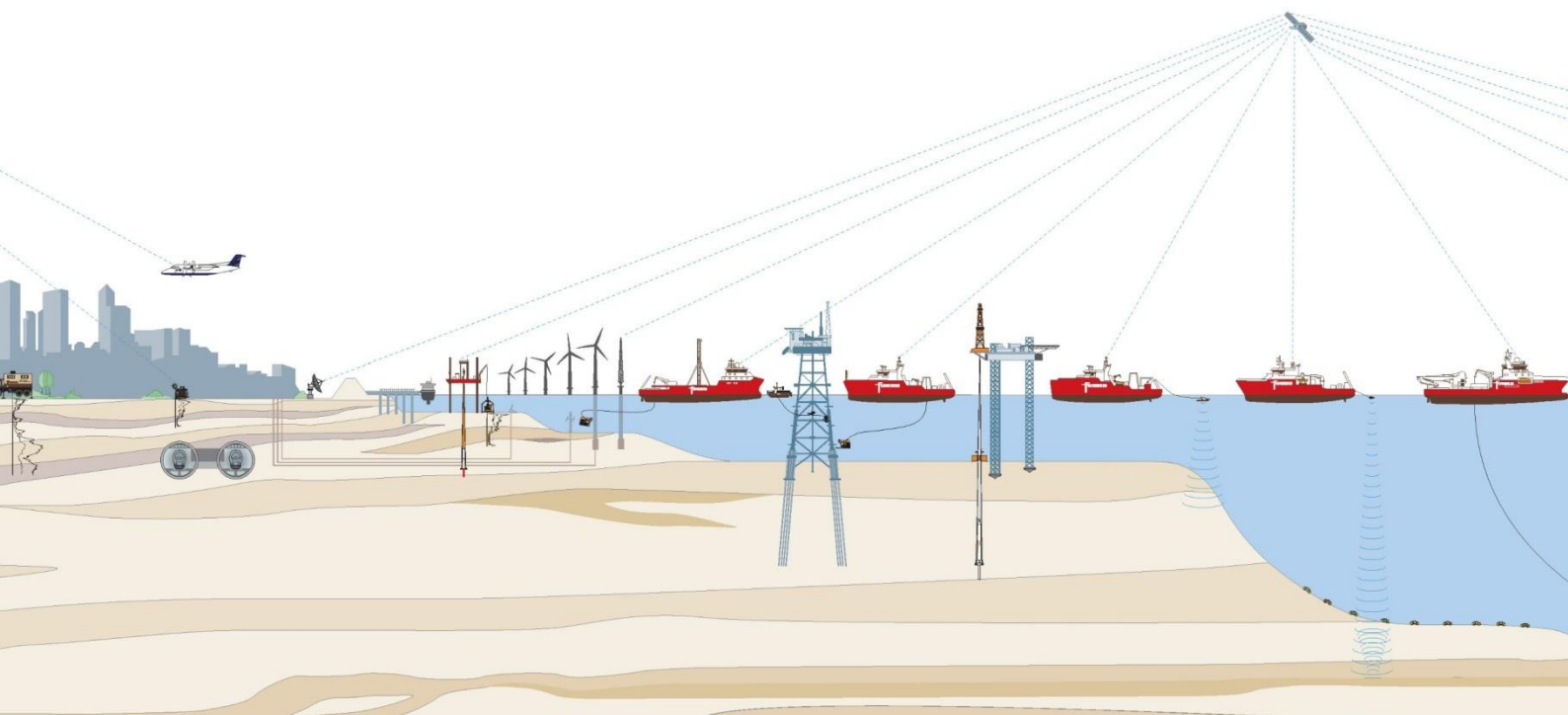


Bay Du Nord Geotechnical Program 2019
Atlantic Ocean, Canada
Geotechnical Report
Volume 3. Data Interpretation and Evaluation of
Representative Geotechnical Parameters

Client Survey ID No. EQ19452
Client Reference No. 20190011-RPT-004
Fugro Document No. P904596/03
Volume 3 of 3
Issue 4



Equinor Canada Ltd.



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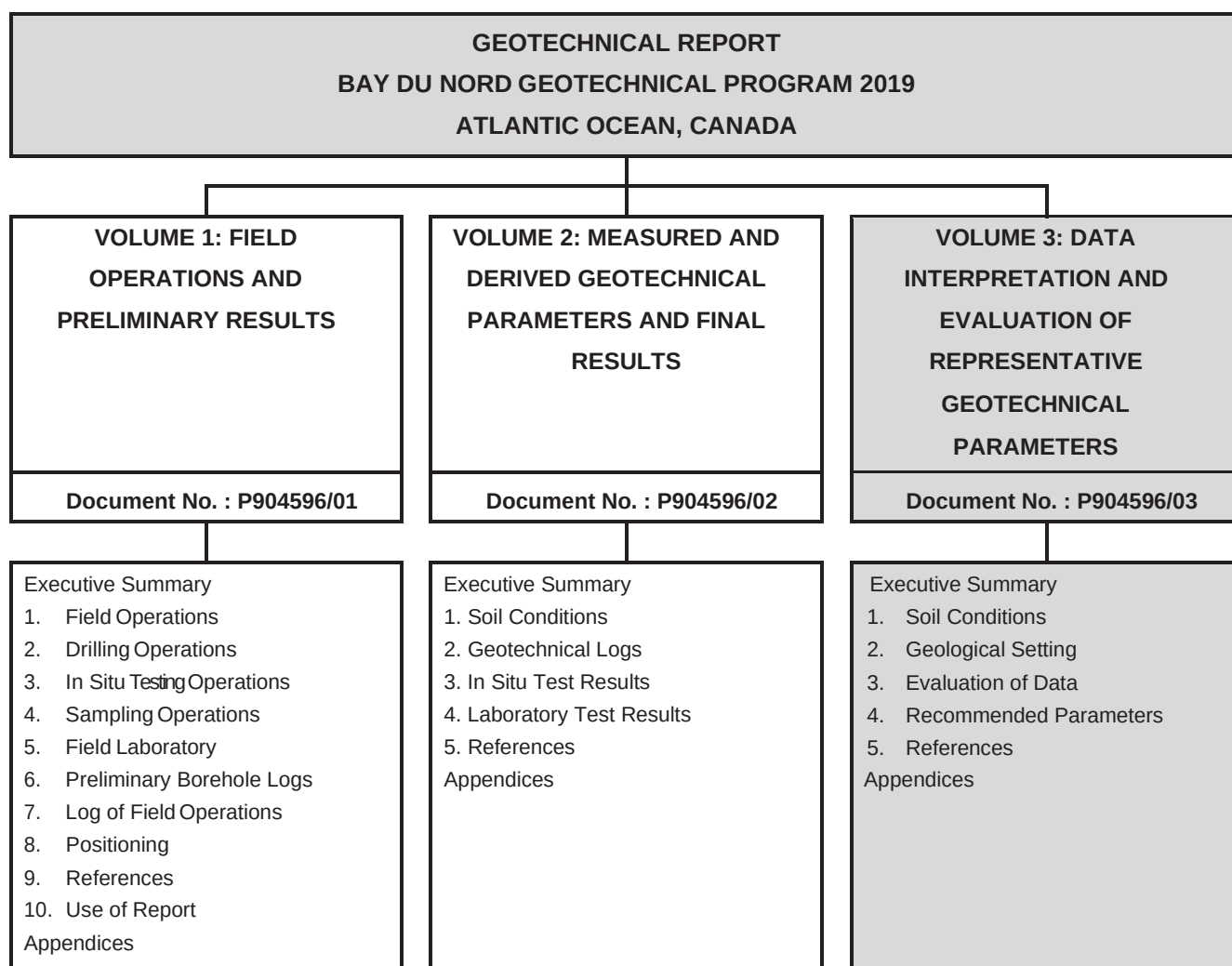
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DOCUMENT ISSUE RECORD

Chapter	Page No.	Plate No.	Issue	Revision
3	27 to 33	3.9-1 to 3.9-9	2	Inclusion of Section “3.9 Cyclic and Dynamic Soil Behaviour”, and associated plates
Abbreviations	x		3	Added terms
1	All		3	Editorials
1		1.1-12 to 1.1-13	3	Editorials and update of “Premise(s)”
2	All		3	Editorials
2		2.2-2	3	Formatting
3	All		3	Editorials
3	18 to 20		3	Sub-sections “3.6.1 Undrained Shear Strength” expanded and “3.6.2 Strength Anisotropy” introduced.
3		3.6-1 to 3.6-5	3	Additional plates: 3.6-1 Undrained Shear Strength versus Net Cone Resistance 3.6-2 Undrained Shear Strength versus Excess Pore Pressure 3.6-3 Cone Factor versus Depth 3.6-4 Cone Factor derived from Pore Pressure Ratio versus Depth 3.6-5 Pore Pressure Factor versus Depth
3		3.8-1, 3.8-1A, 3.8-4	3	Update of data
3		3.9-3 to 3.9-9	3	Editorials, formatting and inclusion of site-specific contour diagrams
4	All		3	Editorials and table added
4		4.2-1	3	Added statistical information
5	All		3	Added references
-	All	-	4	Added hyperlinks to table of contents and list of figures

- 1) The definitive copy of this document is held in Fugro's information system
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- 3) The Issue number of this document is shown on the Document Control page presented immediately after the title page of this document; some (unchanged) pages and plates can show a lower issue number
- 4) The reference at the bottom left-hand corner of each page shows the Fugro document no. and the page issue number (between brackets)

QUALITY MANAGEMENT RECORD

Fugro Project Lead:	E. Beringen Senior Project Engineer
Report Review and Approval:	J. Peuchen Principal Geotechnical Engineer

Document Section / Chapter	Prepared By	Checked By
Executive summary	KSH	EBE
List of Symbols and Terms Used	Fugro/KSH	SJT
Chapter 1 – Soil Conditions	ASA/KSH	EBE
Chapter 2 – Geological Setting	ASA	IBA
Chapter 3 – Evaluation of Data	IBA/JFC/RCN	DRN/MRY/WSO
Chapter 4 – Recommended Parameters	KSH	WSO
Chapter 5 – References	IBA/JFC/RCN/ASA	KSH/EBE
Appendix 1 – Descriptions of Methods and Practices	Fugro	Fugro

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SUMMARY OF MAIN REPORT MODIFICATIONS FROM PREVIOUS REVISION

Sections “3.6.1 Cone Penetration Test Data” and “3.6.2 Laboratory Test Data” have been rearranged into Sections “3.6.1 Undrained Shear Strength” and “3.6.2 Strength Anisotropy”. The following additional plates are introduced in these sub-sections:

- 3.6-1 Undrained Shear Strength versus Net Cone Resistance
- 3.6-2 Undrained Shear Strength versus Excess Pore Pressure
- 3.6-3 Cone Factor versus Depth
- 3.6-4 Cone Factor derived from Pore Pressure Ratio versus Depth
- 3.6-5 Pore Pressure Factor versus Depth

EXECUTIVE SUMMARY

Purpose of Report

Equinor Canada Ltd., is planning to develop the Bay du Nord project in Bay du Nord and Baccalieu fields in the Flemish Pass Basin, offshore eastern Canada, approximately 450 km offshore east-northeast of St. John's, Newfoundland, Canada.

This report presents geotechnical information intended for input in design of subsea templates, FPSO anchors, infield flowlines, riser anchors and pipelines at the Bay du Nord and Baccalieu fields.

Use of Report

This report presents information according to a project specification determined and monitored by Equinor.

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".

Fugro understands that this report will be used for the purpose described above. 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.

Chapter 1 – Soil Conditions

This chapter includes a summary of soil conditions, seafloor conditions and site use.

The soil at the site is normally to slightly overconsolidated clay, divided in three geotechnical units.

Chapter 2 – Geological Setting

This chapter provides a summary of the geological setting and potential geohazards.

The site lies in the northern Flemish Pass in water depths between 1140 m and 1175 m.

The soils at the site were deposited in a hemipelagic environment, showing a relatively high level of lateral continuity.

Chapter 3 – Evaluation of Data

This chapter presents a selection of geotechnical parameters. Parameter value evaluation includes low estimate (LE), best estimate (BE) and high estimate (HE) profiles, as shown below.

Geotechnical Parameter	Profile Type	Chapter Sub-section
Net cone resistance (q_n)	BE	3.3.1
Normalized cone penetration test parameters (I_c , Q_{tn} , F_r)	-	3.3.2
Unit weight (γ)	BE	3.4.1
Water content (w)	-	3.4.2
Liquid limit, plastic limit, plasticity index and liquidity index (w_L , w_P , I_P , I_L)	-	3.4.3
Particle diameter (D_n)	-	3.4.4
Thermal conductivity (k)	-	3.4.5
Overconsolidation ratio (OCR)	BE	3.5.1
Undrained shear strength (s_u)	LE, HE	3.6.1 to 3.6.4
Undrained shear strength of remoulded soil ($s_{u,r}$)	LE, HE	3.6.5
Sensitivity (S_t)	BE	3.6.6
Thixotropy strength ratio (T_x)	-	3.6.7
Angle of interface friction (between ground and foundation) (δ)	-	3.7
Constrained modulus (M)	LE, HE	3.8.1
Coefficient of permeability (k)	LE, HE	3.8.2
Coefficient of consolidation (c_v)	LE, HE	3.8.3
External axial strain at half the maximum deviator stress (ε_{50})	BE	3.8.4
Shear modulus at small strain (G_{max})	BE	3.8.5
Normalised shear modulus (G/G_{max})	-	3.9.1
Cyclic undrained strength parameters (τ_{cy} , τ_{av} , N_f)	Drammen Clay	3.9.2

Data evaluation considers a site-wide approach and assessment per soil unit.

Chapter 4 – Recommended Parameters

This chapter provides an overview, in tabular format, of LE, HE and/or BE profiles presented in Chapter 3.

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SYMBOLS

<u>Symbol</u>	<u>Unit</u>	<u>Quantity</u>
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
g	m/s ²	Acceleration due to gravity ($g = 9.81 \text{ m/s}^2$)
h	m	Height or thickness
i	-	Inclination
m	kg	Mass
t	s	Time
t	a	Time, mean Julian year
v	m/s	Velocity
z	m	Penetration or depth below reference level (usually ground surface)
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 ₀	MPa	Hydrostatic pore pressure relative to seafloor or phreatic surface
u _f	MPa	Pore pressure at failure
Δu	MPa	Change in 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 $[= (\sigma_1 - \sigma_3)/2]$

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	Shear modulus at small strain
I_r	-	Rigidity index [= G/τ_{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' or γ_{sub}	kN/m ³	Unit weight of submerged ground
ρ	Mg/m ³ [= t/m ³]	Density of ground
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 [= $I_D = \gamma_{dmax} (\gamma_d - \gamma_{dmin}) / \gamma_d (\gamma_{dmax} - \gamma_{dmin}) = (e_{max} - e) / (e_{max} - e_{min})$]
v	-	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 [= γ_d / γ_{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 plasticity index 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 [= $(D_{30})^2/D_{10}D_{60}$]
(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	Coefficient of permeability
k_v	m/s	Coefficient of vertical permeability
k_h	m/s	Coefficient of horizontal permeability
i	-	Hydraulic gradient
(f) Thermal and Electrical 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
ρ	$\Omega \cdot m$	Electrical resistivity
K	S/m	Electrical conductivity
(g) Magnetic Properties		
B	T	Magnetic flux density (or magnetic induction)
(h) Radioactive Properties		
γ	CPS	Natural gamma ray
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
Δu	MPa	Excess pore pressure, with respect to hydrostatic
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	-	Normalized cone resistance [$= q_n/\sigma'_{v0}$]
Q_{tn}	-	Normalized cone resistance with variable stress exponent
F_r	%	Normalized friction ratio [$= f_t/q_n$]
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 Δu and s_u
I_c	-	Soil behaviour type index (for Q_{tn} and F_r)
I_{SBT}	-	Soil behaviour type index (for q_c and R_f)

(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
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})
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

Symbol	Unit	Quantity
τ_0	kPa	Initial shear stress
τ_{av}	kPa	Average shear stress or constant shear stress (or τ_a)
τ_{cy}	kPa	Cyclic shear stress amplitude $[= (\tau_{max} - \tau_{min})/2]$
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_r (or external vertical strain at N_r)
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 $\sigma'_v = 100$ kPa during one-dimensional intrinsic compression
e^*_{1000}	-	Void ratio at $\sigma'_v = 1000$ 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	m ² /s	Coefficient of consolidation

Symbol	Unit	Quantity
H	m	Drainage path length
ICL	-	Intrinsic compression line (Burland, 1990)
I_v	-	Void index $[= (e_0 - e^*_{100})/C^*_d]$
m_v	m^2/MN	Coefficient of volume compressibility
M	MPa	Constrained modulus $[= 1/m_v]$
p	kPa	Vertical pressure
OCR	-	Overconsolidation ratio $[= \sigma'_p/\sigma'_{v0}]$ (or yield stress ratio)
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

<u>Symbol</u>	<u>Unit</u>	<u>Quantity</u>
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
z	mm	Axial pile displacement
α	-	Adhesion factor between ground and foundation ($= f/s_u$)
β	-	Adhesion factor between ground and foundation ($= f/\sigma'_v$ 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.

ABBREVIATIONS

I – General

1D	one-dimensional
2D	two-dimensional
3D	three-dimensional
API	American Petroleum Institute
ASTM	American Society for Testing and Materials
BdNBFA	Bay du Nord and Baccalieu Fields Area
BSI	British Standards Institution
COV	coefficient of variation
FEED	front-end engineering design
GIS	geographical information system
ISO	International Organization for Standardization
SD	standard deviation

II – Geodetics

BGL	below ground level
BSF	below seafloor
CD	chart datum
CM	central meridian
DGPS	Differential Global Positioning System
ED	European Datum
ETRS	European terrestrial reference system
GNSS	Global Navigation Satellite System
GPS	Global Positioning System
GRS	Geodetic Reference System
LAT	Lowest Astronomical Tide
MSL	Mean Sea Level
TM	Transverse Mercator
UTC	Coordinated Universal Time
UTM	Universal Transverse Mercator
WGS	World Geodetic System

III – Site Investigation

AUV	autonomous underwater vehicle
BC	box core(r)
BH	borehole
BGL	borehole geophysical logging
BPT	ball penetration test
CPT	cone penetration test
CPTU	piezocone penetration test (or PCPT)
CTD	conductivity temperature depth
FFP	free-fall penetration test
FLPC	Fugro large piston core(r)
GC	gravity core(r)
GS	grab sample(r)
HRS	high resolution seismic reflection
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
MV	motor vessel
PC	piston core(r)
PPDT	pore pressure dissipation test

RC	rotary core(r)
ROV	remotely operated vehicle
SBES	single beam echosounder
SBF	seabed frame
SBP	sub-bottom profiler, seismic reflection
SCPT	seismic cone penetration test
SCS	single channel seismic reflection
SIR	strong impedance reflector
SSDM	Seabed Survey Data Model
SSS	side scan sonar
STACOR®	stationary piston gravity core(r)
SV	sailing vessel
SVP	sound velocity profiler
TPT	T-bar penetration test
TWTT	two-way travel time, seismic reflection
UHRS	ultra high resolution seismic reflection
VC	vibrocore(r)
VST	vane shear test
WISON®	wireline sounding tool
WIP	wireline push sampler

IV – Site Characterisation

BP	before present
Fm	geological formation
LGM	last glacial maximum
Mb	geological formation member
MDAC	methane-derived authigenic carbonate
MTD	mass transport deposit
WLB	with laminae and beds of silt, sand and locally coarser material
UXO	unexploded ordnance

V – Geotechnical Analysis

ALS	accidental limit state
ASD	allowable stress design
FLS	fatigue limit state
LSD	limit state design
PFD	partial factor design
SLS	serviceability limit state
ULS	ultimate limit state
WSD	working stress design

PROJECT SPECIFIC TERMS AND DEFINITIONS

Basic variable: variable representing physical quantities which characterize externally applied loads and environmental influences, geometric quantities, or material properties including soil properties.

Note: Basic variables are typically uncertain random variables or random processes used in the calculation or assessment of representative values of actions or resistance. (ISO 19900:2019)

Best estimate (BE): value of a geotechnical parameter that aims to represent an average or typical derived value for a soil unit, stratum or multiple soil layers.

Calculation model: empirical model, theoretical model, constitutive model, equation or algorithm used for limit state verification.

Note: input for a calculation model typically includes one or more representative values or design values.

Characteristic value: value assigned to a basic variable with a prescribed probability.

Note: In some design/assessment situations, a variable can have two characteristic values, an upper value and a lower value. (ISO 19900:2019)

Derived value: value of a geotechnical parameter obtained by measurement, theory, correlation or empiricism from test results. (Eurocode 7 EN 1997-1:2004)

Design value: value derived from the representative value for use in limit state verification.

Note 1: Design values can be different in different design/assessment situations due to different partial factors. (ISO 19900:2019)

Note 2: The scope of this report excludes design and thus excludes the provision of design values.

Geotechnical parameter: basic variable representing a soil property.

Note: examples of geotechnical parameters are undrained shear strength and net cone resistance.

High estimate (HE): value of a geotechnical parameter that aims to represent a high derived value for a soil unit, stratum or multiple soil layers.

Limit state verification: demonstration that the total design action effect in each design/assessment situation does not exceed the limit state design resistance. (ISO 19900:2019)

Low estimate (LE): value of a geotechnical parameter that aims to represent a low derived value for a soil unit, stratum or multiple soil layers.

Nominal value: value assigned to a variable specified or determined on a non-statistical basis, typically from acquired experience or physical conditions, or as published in a recognized code or standard.

Note: In some design/assessment situations, a variable can have two nominal values, an upper value and a lower value. (ISO 19900:2019)

Recommended representative geotechnical parameter: geotechnical parameter that is common for one or more calculation models typically used for the design of the offshore structure(s) under consideration.

Representative value: value assigned to a basic variable for verification of a limit state in a design/assessment situation.

Note: Two types of representative value used in design verification are characteristic value and nominal value. (ISO 19900:2019)

Site: geographical part of the earth at a particular time or period, that:

- Incorporates the surface of the earth and ground below this surface;
- Can include water and air above the ground surface;
- Can include man-made objects and structures in or connected to the ground.

CHAPTER 1 – SOIL CONDITIONS

1. SOIL CONDITIONS

1.1 Introduction

This chapter presents a summary of soil conditions for the site (Plate 1.1-2). Chapter 2 presents further information about the geological setting.

The term “Bay du Nord and Baccalieu Fields Area” (BdNBFA) refers to the area that includes the geotechnical program 2019 campaign (Volume 2, Fugro, 2019a). In this report, it is used to define the area shown on Plate 1.1-2.

The term “location” refers to a place at seafloor where a geotechnical borehole was performed.

The term “site” in this report is composed of the following sub-sites considered for the Bay du Nord Geotechnical Program 2019 (Plate 1.1-2):

- “template XXXX sub-site” referring to an area defined by 50 m radius circle centred in the middle of the geotechnical borehole location group at a proposed template position, with a vertical extent to 31.8 m below seafloor;
- The term “FPSO anchor sub-site” referring to an area defined by 10 m radius circle, encompassing a proposed FPSO anchor position, with a vertical extent to 31.8 m below seafloor.

Plates 1.1-12 and 1.1-13 present the study approach for data interpretation and evaluation of representative geotechnical parameters, including seafloor soil conditions and general stratigraphy (this chapter) and information presented in Chapters 2 to 4. The section titled “Sources of Information” (below) lists where further details can be found.

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”.

Fugro understands that this report will be used for the purpose described on Plates 1.1-12 and 1.1-13. 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.

1.2 Seafloor Conditions and Site Use

This report section and Table 1.1 present seafloor conditions for the site.

Table 1.1: Seafloor Conditions

Seafloor Conditions in Sub-sites	Description
Topography	Seafloor is essentially flat across each sub-site; water depth varies less than 1 m within each sub-site. Water depth is between 1165 m to 1170 m MSL at the anchor and the North, Central and South Template sub-sites. Water depth is approximately 1140 m MSL at the Baccalieu Template sub-site. Footprints and debris (i.e. drill cuttings) from geotechnical investigation activities may be present at the borehole locations.
Gradient	Slope of <1.5° within each sub-site. Seafloor dips slightly west.
Soil type(s) at seafloor	Extremely low strength slightly sandy to sandy, slightly gravelly CLAY.

Seafloor Conditions in Sub-sites	Description
Objects	No evidence of anthropogenic objects observed within the sub-sites. A sonar contact interpreted as a possible dropstone was observed within one sub-site, in the Template South sub-site, approximately 20 m southwest of geotechnical location EQ19452-BH-24.

The available data show no evidence of anthropogenic structures at the site. Hydrocarbon wells are at least 800 m from each sub-site. The closest is Bay du Nord L-76, which is 0.8 km NW of the Template South site EQ19452-BH-21. The nearest subsea cable is charted outside BdNBFA, more than 2 km south of the South Template sub-site.

Anthropogenic activity at the site comprised marine soil investigations (Geotechnical Program 2019) and, probably, fishing with bottom trawling equipment.

The BdNBFA shows local seafloor features described as pockmarks and hummocks or mounds, with some interpreted as fluid escape features (Plates 1.1-2 to 1.1-7). However, the available data show no evidence for these features at the site. In this regard it should be noted that the available, relevant, marine geophysical investigation data cover most but not all of the site.

Fugro (2019 and 2020b) indicate sonar contacts which may be interpreted as boulders. Small objects, such as cobbles, typically remain undetected by marine geophysical investigation (ISO, 2018).

1.3 Stratigraphy and Spatial Variability

Table 1.2 presents the general stratigraphy for the template and anchor sub-sites, based on the depths of occurrence at the geotechnical borehole locations.

The stratigraphy is assessed to be generally laterally continuous, with local variation in unit character and thickness). The stratigraphy agrees with previous investigations and the geological framework for this area of the North Atlantic.

The primary soil type is clay. Subordinate bedding is present as silt, sand and locally coarser material. These laminae and beds are typically too thin to be described as individual strata. This report includes the abbreviation WLB (with laminae and beds of silt, sand and locally coarser material). Chapter 2 includes explanations for WLB occurrence.

Table 1.2: General Site Stratigraphy

Unit	Depth to Top of Unit [m] BSF	Depth to Base of Unit [m] BSF	Description	Spatial Variability
I	0.0	2.0 to 3.8	Extremely low strength, silty, slightly sandy CLAY Locally slight sandy to sandy, slightly gravelly	Assessed as present in at all geotechnical locations and across the site, with local variation of thickness.
II	2.0 to 3.8	27.1 to >31.8	Very low strength to medium strength, silty slightly sandy, slightly gravelly CLAY Locally: ■ High strength below 23 m BSF ■ laminae to medium beds of silt and sand ■ with traces of shell fragments ■ with occasional cobbles	Assessed as present in at all geotechnical locations and across the site, with local variation of thickness.
III	27.1 to >31.8	-	Medium strength to high strength, silty slightly gravelly CLAY	Expected to be present across site, though at some locations, top of unit below depth of investigation (31.8 m BSF)
Notes: - > = greater than - Information is presented for depth range of interest - Base of Unit II is variable around approximately 30 m BSF - Base of Unit III is below 220 m BSF				

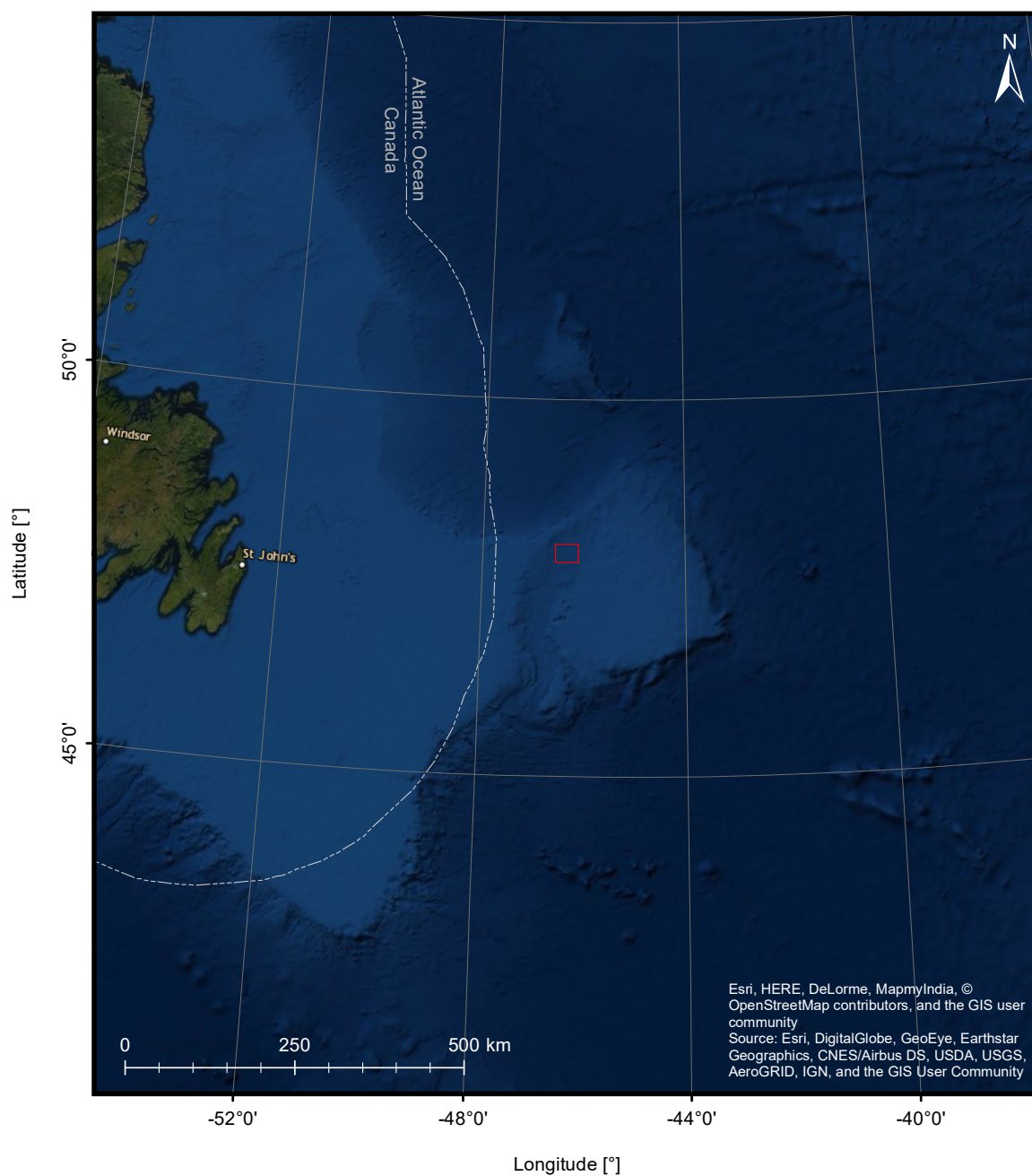
1.4 Sources of Information

This report uses and summarises selected information, where appropriate;

- Geotechnical data presented in Volumes 1 and 2 of this report (refer to cover page titled “Geotechnical Report, Bay du Nord Geotechnical Program 2019, Atlantic Ocean, Canada”; referenced as Fugro, 2019a and Fugro, 2020a respectively);
- Client-provided geotechnical data including site investigation data (Benthic, 2015; NGI, 2016a and 2016b);
- Geophysical information:
 - a purpose-specific 2018 geophysical investigation: sub-bottom and bathymetry data and a report including assessments of seafloor features and seabed stratigraphy (Fugro, 2019b);
 - previous marine geophysical studies (Fugro, 2013, 2014a, 2014b, 2015, 2016, 2019c-f)
 - a subsurface boulder assessment (Fugro, 2020b);
- Site-specific desktop geohazard study (Fugro, 2017)
- Client-provided site-specific seismic hazard analysis (Golder Associates, 2014)
- Publicly available information.

List of Plates

	Plate
Vicinity Map	1.1-1
Detailed Location Plan	1.1-2 to 1.1-7
Coordinates and Water Depth	1.1-8 to 1.1-11
Study Approach	1.1-12 to 1.1-13



Approximate area of investigation

Maritime boundary

Datum : North American 1983

Ellipsoid : GRS 1980

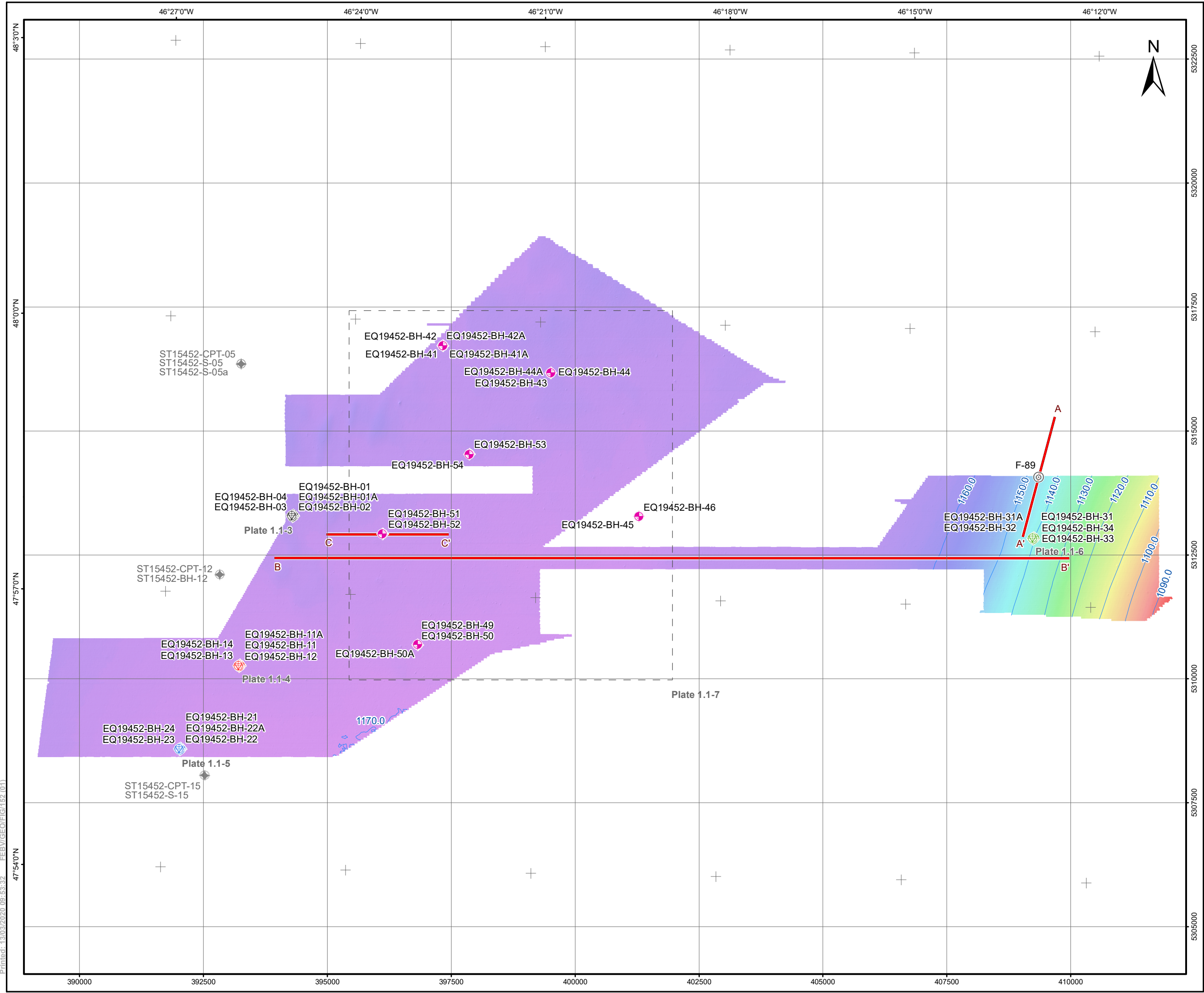
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Central Meridian: 45° West (UTM Zone 23 N)

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VICINITY MAP

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GEOTECHNICAL LOCATIONS

Template North - Plate 1.1-3

- Rotary borehole drilling and sampling
- Rotary borehole drilling and testing

Template Central - Plate 1.1-4

- Rotary borehole drilling and sampling
- Rotary borehole drilling and testing

Template South - Plate 1.1-5

- Rotary borehole drilling and sampling
- Rotary borehole drilling and testing

Template Baccalieu - Plate 1.1-6

- Rotary borehole drilling and sampling
- Rotary borehole drilling and testing

FPSO Anchors - Plate 1.1-7

- Rotary borehole drilling and sampling
- Rotary borehole drilling and testing

Client Provided Site Investigation Data

- Investigation location

GENERAL

- Well
- Profile line
- Plate extent indicator

BATHYMETRY

- Depth contour at 10.0 metres interval

Water Depth
[m below MSL]

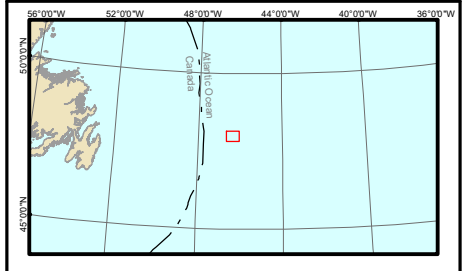
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1171.0

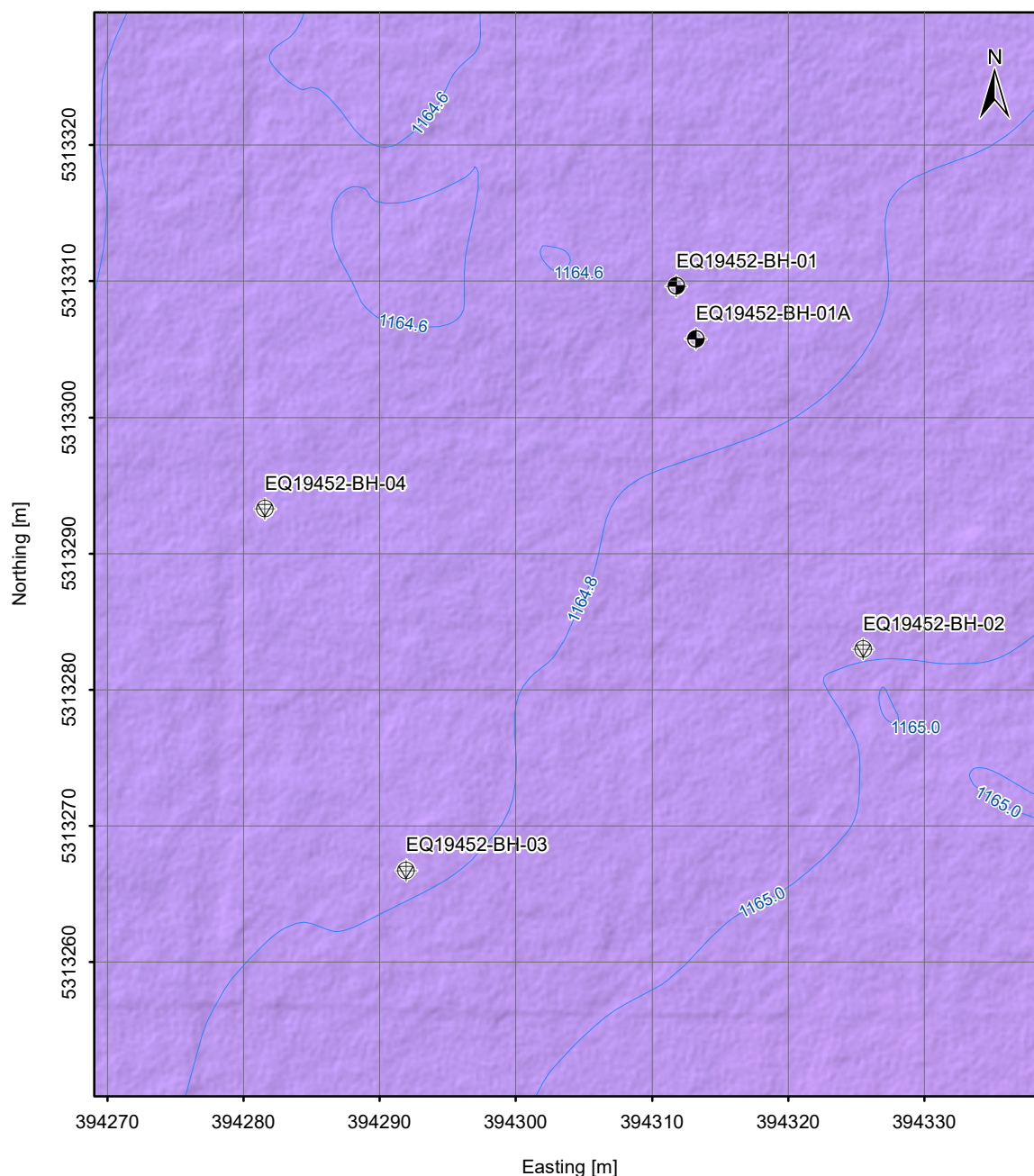
NOTES

- Source bathymetry data: Fugro, 2018
- Source client provided site investigation data: NGI, 2016a

GEODETC PARAMETERS

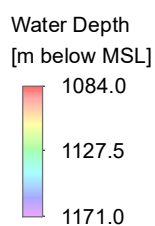
EPSG Code	26923
DATUM	North American 1983
Ellipsoid	GRS 1980
Semi major axis	a = 6 378 137.000
Inverse flattening	1/f = 298.257222101
PROJECTION	UTM, Zone 23 North
Central Meridian (CM)	45° 00' 00" W
Latitude of Origin	0° 00' 00" N
False Easting	500 000 m
False Northing	000 000 m
Scale factor	0.9996
Units	metres / degrees





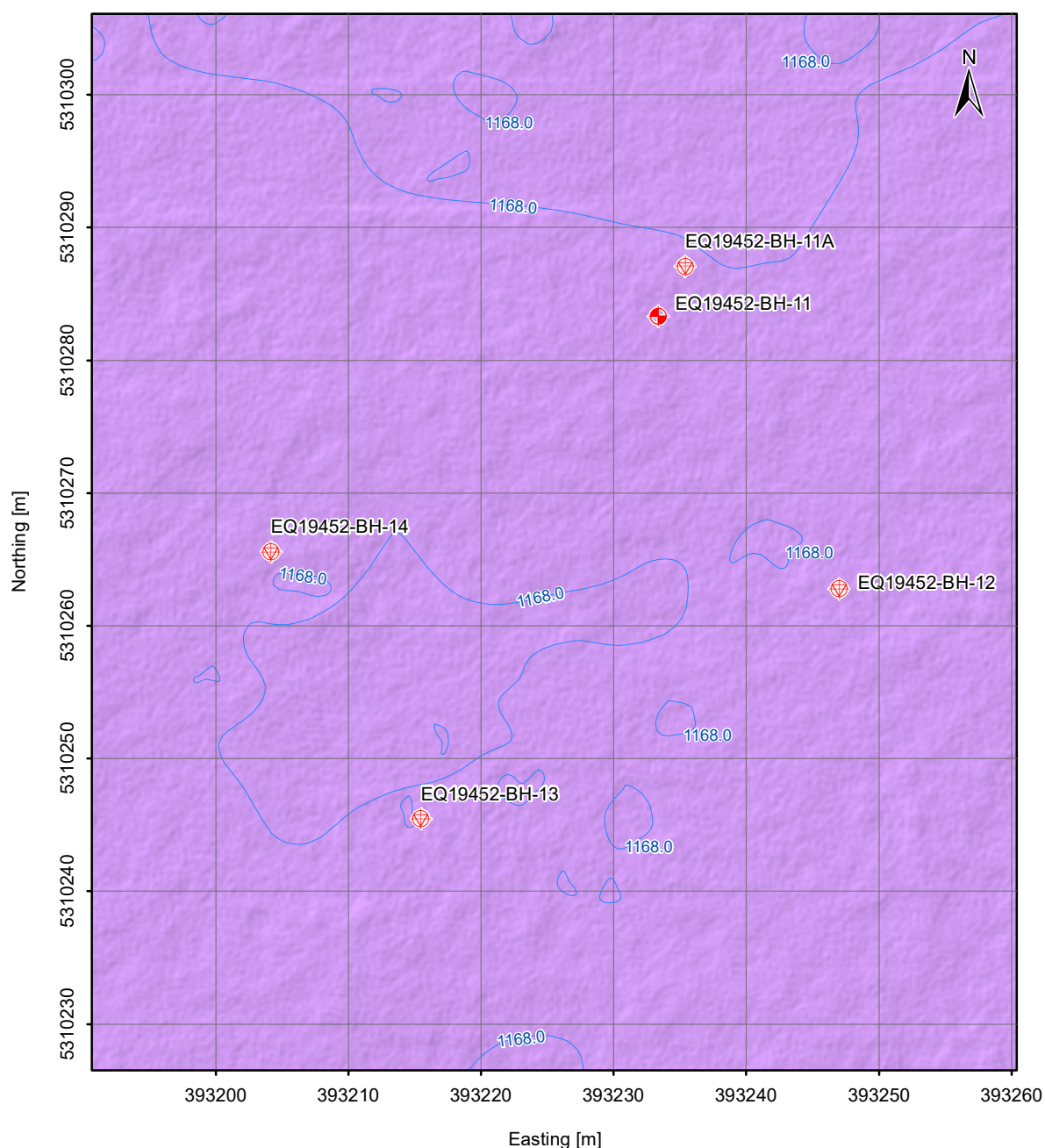
-  Rotary borehole drilling and sampling
-  Rotary borehole drilling and testing
-  Depth contour at 0.2 metres interval

Datum : North American 1983
 Ellipsoid : GRS 1980
 Projection : Universal Transverse Mercator (UTM)
 Central Meridian : 45° West (UTM Zone 23 N)



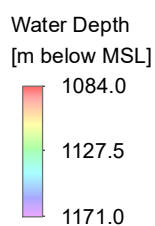
DETAILED LOCATION PLAN
 TEMPLATE NORTH

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-  Rotary borehole drilling and sampling
-  Rotary borehole drilling and testing
-  Depth contour at 0.2 metres interval

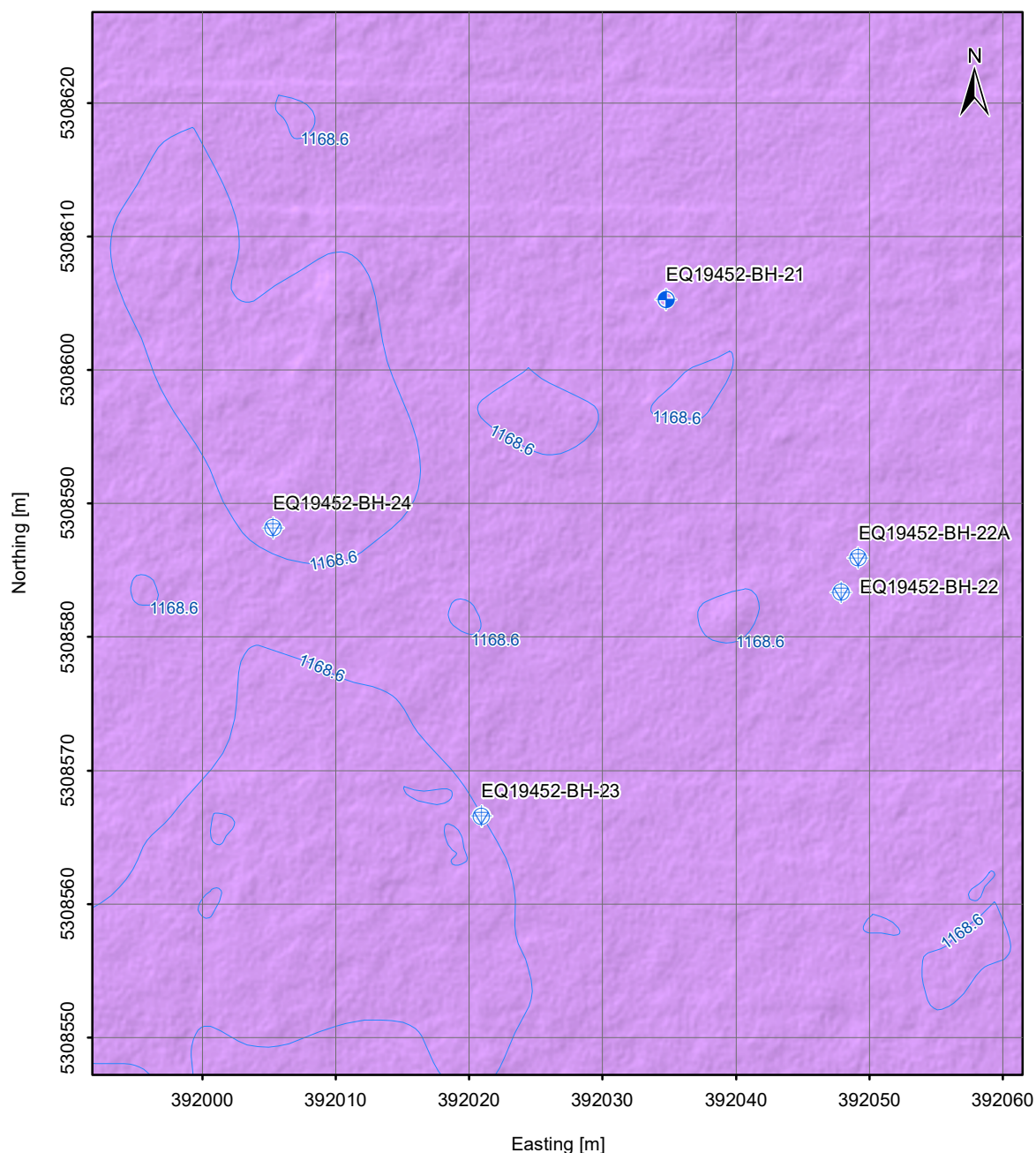
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 Ellipsoid : GRS 1980
 Projection : Universal Transverse Mercator (UTM)
 Central Meridian : 45° West (UTM Zone 23 N)



DETAILED LOCATION PLAN

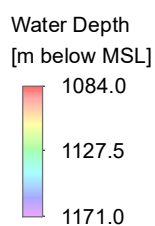
TEMPLATE CENTRAL

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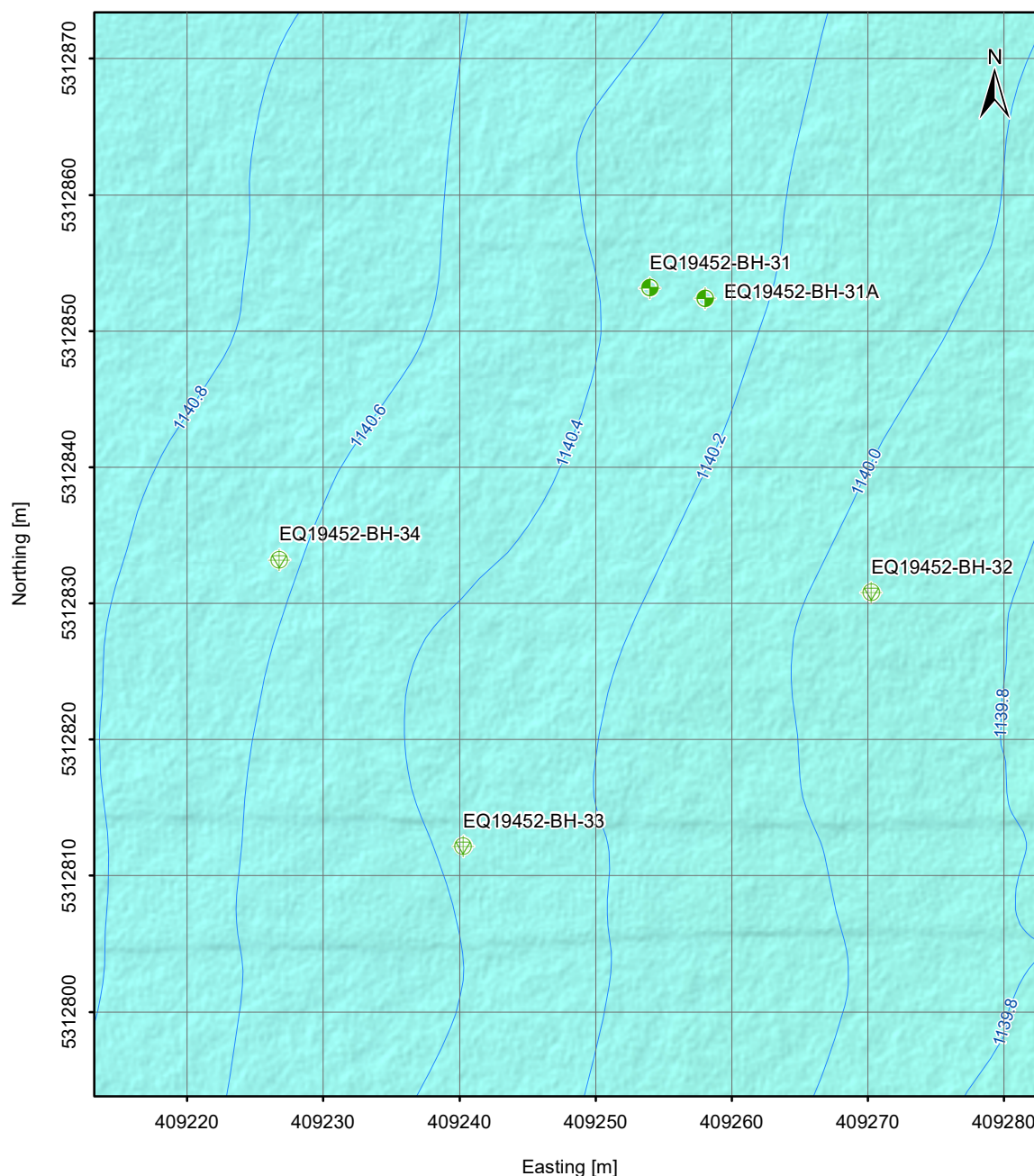
-  Rotary borehole drilling and sampling
-  Rotary borehole drilling and testing
-  Depth contour at 0.2 metres interval

Datum : North American 1983
 Ellipsoid : GRS 1980
 Projection : Universal Transverse Mercator (UTM)
 Central Meridian : 45° West (UTM Zone 23 N)



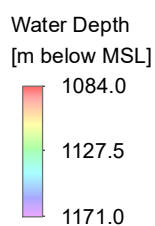
DETAILED LOCATION PLAN TEMPLATE SOUTH

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-  Rotary borehole drilling and sampling
-  Rotary borehole drilling and testing
-  Depth contour at 0.2 metres interval

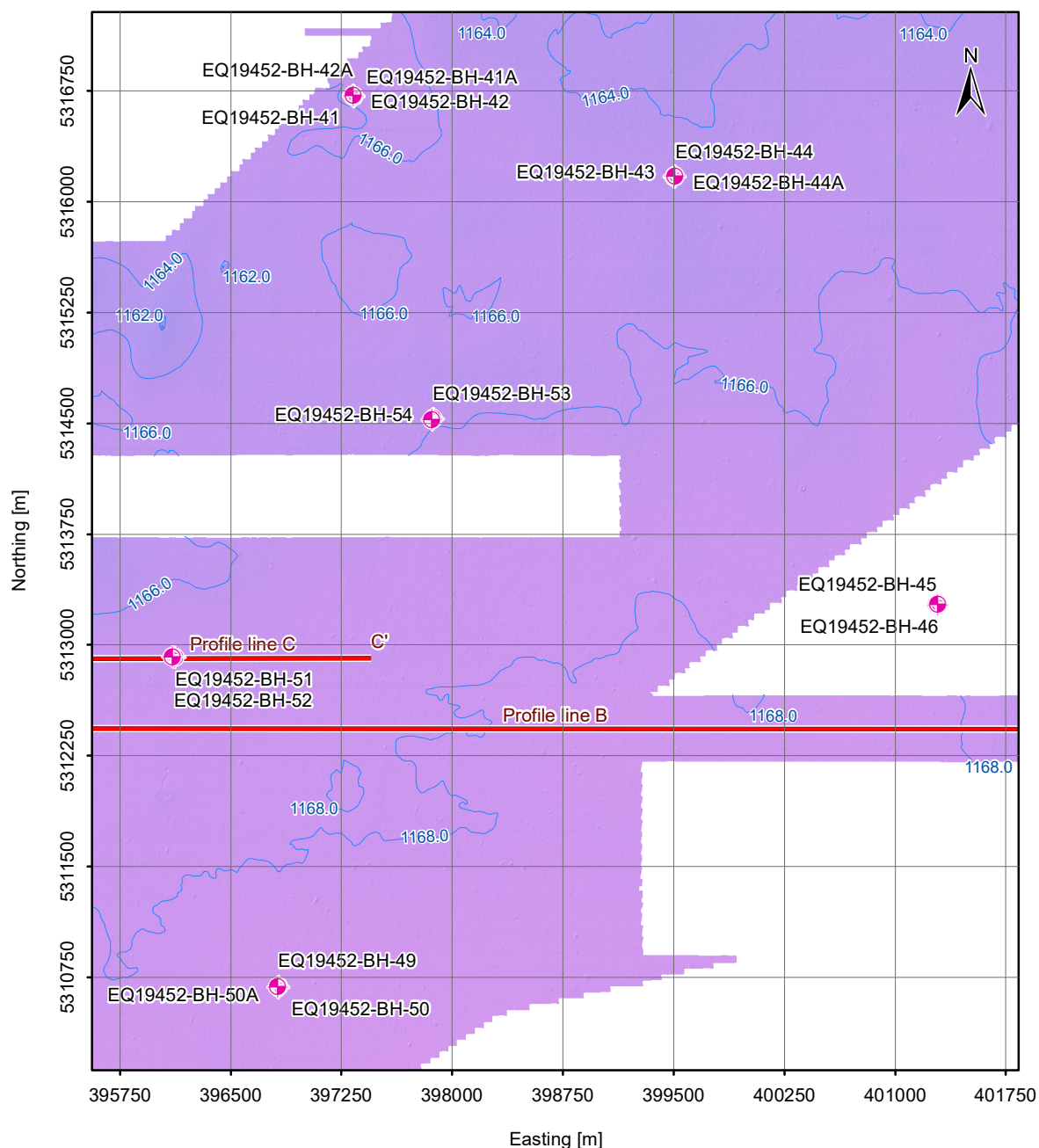
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 Ellipsoid : GRS 1980
 Projection : Universal Transverse Mercator (UTM)
 Central Meridian : 45° West (UTM Zone 23 N)



DETAILED LOCATION PLAN

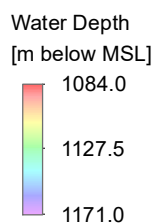
TEMPLATE BACCALIEU

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-  Rotary borehole drilling and sampling
-  Rotary borehole drilling and testing
-  Profile line
-  Depth contour at 2.0 metres interval

Datum : North American 1983
 Ellipsoid : GRS 1980
 Projection : Universal Transverse Mercator (UTM)
 Central Meridian : 45° West (UTM Zone 23 N)



DETAILED LOCATION PLAN

FPSO ANCHORS

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BAY DU NORD GP 2019 - ATLANTIC OCEAN, CANADA

COORDINATES					
Location	Penetration Depth [m]	Easting [m]	Northing [m]	Latitude [deg]	Longitude [deg]
EQ19452-BH-01	9.0	394312	5313310	47°57'51.7" N	46°24'57.1" W
EQ19452-BH-01A	31.5	394313	5313306	47°57'51.6" N	46°24'57.0" W
EQ19452-BH-02	32.0	394326	5313283	47°57'50.9" N	46°24'56.4" W
EQ19452-BH-03	32.0	394292	5313267	47°57'50.3" N	46°24'58.0" W
EQ19452-BH-04	30.0	394282	5313293	47°57'51.2" N	46°24'58.5" W
EQ19452-BH-11	30.5	393233	5310283	47°56'13.1" N	46°25'46.4" W
EQ19452-BH-11A	3.0	393235	5310287	47°56'13.2" N	46°25'46.3" W
EQ19452-BH-12	32.0	393247	5310263	47°56'12.4" N	46°25'45.7" W
EQ19452-BH-13	29.4	393215	5310245	47°56'11.8" N	46°25'47.2" W
EQ19452-BH-14	30.5	393204	5310266	47°56'12.5" N	46°25'47.8" W
Datum : North American 1983 Projection : UTM Zone 23 N Spheroid : GRS 1980 Central Meridian : -45° East Penetration Depth: deepest point reached by drilling, sampling or in situ testing, relative to seafloor For further details, refer to Chapter 8 "Positioning", presented in Volume 1 Report (Fugro, 2019)					

WATER DEPTH					
Location	Date	CTD Probe Reduced [m]	Drill String Reduced [m]	USBL Reduced [m]	Bathymetry Reduced [m]
EQ19452-BH-01	11-Jul-2019	1164.7	1164.7	1166.4	1164.7
EQ19452-BH-01A	12-Jul-2019	1164.7	1164.6	1166.6	1164.8
EQ19452-BH-02	09-Jul-2019	1165.0	1164.8	1166.6	1165.0
EQ19452-BH-03	10-Jul-2019	1164.7	1164.7	1166.3	1164.8
EQ19452-BH-04	11-Jul-2019	1164.6	1164.5	1166.5	1164.7
EQ19452-BH-11	26-Jul-2019	1167.9	1167.6	1168.9	1168.0
EQ19452-BH-11A	26-Jul-2019	1168.1	1167.5	1168.9	1168.0
EQ19452-BH-12	23-Jul-2019	1167.9	1167.8	1169.0	1167.9
EQ19452-BH-13	24-Jul-2019	1168.0	1167.7	1169.0	1168.0
EQ19452-BH-14	24-Jul-2019	1167.9	1167.6	1168.8	1168.0
Reduced water depths are relative to Mean Sea Level					

COORDINATES AND WATER DEPTH

COORDINATES					
Location	Penetration Depth [m]	Easting [m]	Northing [m]	Latitude [deg]	Longitude [deg]
EQ19452-BH-21	29.5	392035	5308605	47°55'18.0" N	46°26'42.6" W
EQ19452-BH-22	3.0	392048	5308583	47°55'17.3" N	46°26'42.0" W
EQ19452-BH-22A	32.0	392049	5308586	47°55'17.4" N	46°26'41.9" W
EQ19452-BH-23	30.0	392021	5308567	47°55'16.8" N	46°26'43.2" W
EQ19452-BH-24	30.5	392005	5308588	47°55'17.5" N	46°26'44.0" W
EQ19452-BH-31	14.0	409254	5312853	47°57'45.2" N	46°12'56.3" W
EQ19452-BH-31A	30.0	409258	5312852	47°57'45.2" N	46°12'56.1" W
EQ19452-BH-32	30.0	409270	5312831	47°57'44.5" N	46°12'55.5" W
EQ19452-BH-33	30.0	409240	5312812	47°57'43.9" N	46°12'56.9" W
EQ19452-BH-34	30.0	409227	5312833	47°57'44.5" N	46°12'57.6" W
Datum : North American 1983 Projection : UTM Zone 23 N Spheroid : GRS 1980 Central Meridian : -45° East Penetration Depth: deepest point reached by drilling, sampling or in situ testing, relative to seafloor For further details, refer to Chapter 8 "Positioning", presented in Volume 1 Report (Fugro, 2019)					

WATER DEPTH					
Location	Date	CTD Probe Reduced [m]	Drill String Reduced [m]	USBL Reduced [m]	Bathymetry Reduced [m]
EQ19452-BH-21	29-Jul-2019	1168.7	1168.6	1170.2	1168.6
EQ19452-BH-22	29-Jul-2019	1168.5	1168.4	1170.4	1168.7
EQ19452-BH-22A	30-Jul-2019	1168.6	1168.5	1170.3	1168.6
EQ19452-BH-23	31-Jul-2019	1168.6	1168.3	1170.5	1168.6
EQ19452-BH-24	28-Jul-2019	1168.6	1168.4	1170.4	1168.5
EQ19452-BH-31	08-Aug-2019	1140.0	1140.8	1142.9	1140.4
EQ19452-BH-31A	09-Aug-2019	1139.9	1140.8	1142.9	1140.3
EQ19452-BH-32	06-Aug-2019	1139.4	1140.7	1142.4	1139.9
EQ19452-BH-33	07-Aug-2019	1139.8	1140.4	1142.9	1140.4
EQ19452-BH-34	07-Aug-2019	1140.2	1140.6	1143.2	1140.6
Reduced water depths are relative to Mean Sea Level					

COORDINATES AND WATER DEPTH

BAY DU NORD GP 2019 - ATLANTIC OCEAN, CANADA

COORDINATES					
Location	Penetration Depth [m]	Easting [m]	Northing [m]	Latitude [deg]	Longitude [deg]
EQ19452-BH-41	9.0	397333	5316719	47°59'43.9" N	46°22'34.4" W
EQ19452-BH-41A	30.0	397333	5316713	47°59'43.7" N	46°22'34.4" W
EQ19452-BH-42	30.0	397326	5316722	47°59'44.0" N	46°22'34.7" W
EQ19452-BH-42A	5.0	397329	5316723	47°59'44.0" N	46°22'34.5" W
EQ19452-BH-43	30.0	399504	5316171	47°59'27.4" N	46°20'49.2" W
EQ19452-BH-44	29.0	399512	5316173	47°59'27.4" N	46°20'48.8" W
EQ19452-BH-44A	29.0	399509	5316175	47°59'27.5" N	46°20'48.9" W
EQ19452-BH-45	30.0	401282	5313275	47°57'54.6" N	46°19'21.0" W
EQ19452-BH-46	30.0	401289	5313279	47°57'54.7" N	46°19'20.7" W
EQ19452-BH-49	30.0	396822	5310690	47°56'28.4" N	46°22'53.8" W
Datum : North American 1983 Projection : UTM Zone 23 N Spheroid : GRS 1980 Central Meridian : -45° East Penetration Depth: deepest point reached by drilling, sampling or in situ testing, relative to seafloor For further details, refer to Chapter 8 "Positioning", presented in Volume 1 Report (Fugro, 2019)					

WATER DEPTH					
Location	Date	CTD Probe Reduced [m]	Drill String Reduced [m]	USBL Reduced [m]	Bathymetry Reduced [m]
EQ19452-BH-41	14-Aug-2019	1165.2	1165.5	1168.0	1166.1
EQ19452-BH-41A	15-Aug-2019	1165.1	1165.1	1167.8	1166.1
EQ19452-BH-42	16-Aug-2018	1165.2	1165.7	1167.9	1166.2
EQ19452-BH-42A	16-Aug-2019	1165.2	1165.7	1167.8	1166.1
EQ19452-BH-43	17-Aug-2019	1163.9	1164.3	1166.3	1164.8
EQ19452-BH-44	19-Aug-2019	1163.8	1164.3	1166.1	1164.7
EQ19452-BH-44A	19-Aug-2019	1163.9	1164.2	1166.0	1164.7
EQ19452-BH-45	10-Aug-2019	1167.5	1167.8	1170.4	-
EQ19452-BH-46	11-Aug-2019	1167.4	1167.9	1170.4	-
EQ19452-BH-49	12-Aug-2019	1168.2	1168.6	1163.9	1168.7
Reduced water depths are relative to Mean Sea Level					

COORDINATES AND WATER DEPTH

COORDINATES					
Location	Penetration Depth [m]	Easting [m]	Northing [m]	Latitude [deg]	Longitude [deg]
EQ19452-BH-50	31.5	396823	5310683	47°56'28.1" N	46°22'53.8" W
EQ19452-BH-50A	30.0	396817	5310685	47°56'28.2" N	46°22'54.0" W
EQ19452-BH-51	30.0	396121	5312920	47°57'40.2" N	46°23'29.5" W
EQ19452-BH-52	22.0	396108	5312922	47°57'40.2" N	46°23'30.1" W
EQ19452-BH-53	30.0	397869	5314534	47°58'33.4" N	46°22'6.6" W
EQ19452-BH-54	30.0	397865	5314528	47°58'33.2" N	46°22'6.8" W
Datum : North American 1983 Projection : UTM Zone 23 N Spheroid : GRS 1980 Central Meridian : -45° East Penetration Depth: deepest point reached by drilling, sampling or in situ testing, relative to seafloor For further details, refer to Chapter 8 "Positioning", presented in Volume 1 Report (Fugro, 2019)					

WATER DEPTH					
Location	Date	CTD Probe Reduced [m]	Drill String Reduced [m]	USBL Reduced [m]	Bathymetry Reduced [m]
EQ19452-BH-50	13-Aug-2019	1168.2	1168.7	1163.9	1168.7
EQ19452-BH-50A	13-Aug-2019	1168.1	1168.7	1163.8	1168.7
EQ19452-BH-51	22-Aug-2019	1166.7	1167.2	1169.6	1167.0
EQ19452-BH-52	23-Aug-2019	1166.6	1167.3	1169.5	1166.9
EQ19452-BH-53	20-Aug-2019	1165.5	1165.9	1167.9	1165.9
EQ19452-BH-54	21-Aug-2019	1165.5	1166.0	1167.8	1165.9
Reduced water depths are relative to Mean Sea Level					

COORDINATES AND WATER DEPTH

STUDY APPROACH

General Procedure:	– Refer to documents titled "Site Characterisation" and "Geotechnical Analysis" presented in appendix titled "Descriptions of Methods and Practices"
Premise(s):	– According to ISO 19900 (2019); ISO 19901-4 (2016) – Applicable project phases are "conceptual design", "FEED" (front end engineering design) and "detailed design" of structure(s) – Presented information is project-specific and depends on project phase, structure type(s) and purpose – Verification of the presented information is recommended prior to final installation decisions
Type of Structure(s) and Purpose:	Subsea templates, FPSO anchors, pipelines, infield flowlines and/or riser anchors
Location:	Refer to Executive Summary for definition of site location and site area
Integration Level for Site Characterisation:	Level 4 Geotechnical Zonation and Analysis
Site Use:	Refer to Chapters 1 to 4 and document titled "Site Characterisation" in appendix titled "Descriptions of Methods and Practices"
Seafloor Conditions and Hazards:	Refer to Chapter 1 plate(s) titled "Site Conditions"
Stratigraphy and Ground Parameters:	Refer to Chapters 1 and 2
	Refer to Chapters 1 to 4 and Appendices

DATA COVERAGE

Information on Historic and Current Site Use:	– Refer to Chapter 1 plate(s) titled "Site Conditions"
Seismic (Earthquake) Data:	– Available, refer to Chapters 1 and 2 – Site-specific seismic hazard analysis (Golder Associates, 2014) – Seismic zonation according to ISO (2017)
Geological Data:	Available, refer to Chapters 1 and 2
Geophysical Survey Data:	Available, refer to Chapters 1 and 2
Geotechnical Data:	Available, refer to Main Text and plate(s) titled "Detailed Location Plan"

INFORMATION MANAGEMENT

Data Processing:	– GeODin® software for geotechnical data processing and presenting – UNIPLOT software for in situ test data – General purpose software, including ArcGIS
Data Format(s) for Results:	PDF for viewing and printing (this primary document)

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STUDY APPROACH

REFERENCES

- Computer Program ESRI ArcGIS, Analysis and Presentation of Geo-data, Version 10.3
- Computer Program GeODin®, Recording, Presentation and Analysis of Geo-data
- Computer Program UNIPLOT, Processing, Presentation and Analysis of In Situ Test Data
- Golder Associates, 2014. Site-specific seismic hazard analysis - Flemish Pass Project Site, St. John's: Golder Associates. Golder Associates, 2014. Site-specific seismic hazard analysis - Flemish Pass Project Site, St. John's: Golder Associates.
- International Organization for Standardization, 2019. ISO 19900:2019 Petroleum and Natural Gas Industries - General Requirements for Offshore Structures. Geneva: ISO.
- International Organization for Standardization, 2017. ISO 19901-2:2017 Petroleum and natural gas industries: specific requirements for offshore structures — Part 2: Seismic design procedures and criteria. Geneva: ISO.
- International Organization for Standardization, 2016. ISO 19901-4:2016 Petroleum and Natural Gas Industries - Specific Requirements for Offshore Structures - Part 4: Geotechnical and Foundation Design Considerations. Geneva: ISO.

CHAPTER 2 – GEOLOGICAL SETTING

2. GEOLOGICAL SETTING

2.1 Introduction

This chapter about the geological setting and potential geohazards is based on the sources of information listed above in Chapter 1 and listed in Chapter 5 “References”.

Plates 2.2-1, 2.2-2, and 2.2-3 display examples of regional stratigraphy in the BdNBFA on cross-sections of geophysical data. The transect locations (A-A', B-B' and C-C') are illustrated on Plate 1.1-2.

2.2 Geology and Seismo-stratigraphy

The BdNBFA is in the northern Flemish Pass. The pass separates pieces of the south-easternmost continental shelf of Canada (Plate 1.1-1), the Grand Banks on the west and the Flemish Cap on the east, through a fault which has not been active in the last 50 million years. The pass ranges from 40 km to 100 km in width. Water depth in the Flemish Pass reaches 1500 m, while the water depth over the continental shelf immediately on either side ranges from 100 m to 250 m.

The soils observed in the Geotechnical Program 2019 are as expected in this setting and consistent with previous studies. That is, deposits within the site are comprised predominantly of clay. They were likely deposited in primarily low energy, hemipelagic conditions. The upper portion of Unit I is an extremely low strength to very low strength clay, which has been observed and described as regional drape.

The occasional thin sand and silt layers of Units I, II and III are likely the deposits of the remaining material of mass transport events originating on the shelf, the Grand Banks or Flemish Cap.

Occasional ice-rafted deposits appear to be present. At some geotechnical locations, occasional gravel and cobbles were observed. These have been interpreted as ice-rafted deposits. The soils within the site show no evidence of ice loading.

Plates 2.2-1, 2.2-2, and 2.2-3 present seismic data which illustrate the general lateral continuity, i.e. the relative lack of horizontal spatial variability, of Units I and II. Plate 2.1-1 is reproduced from a report assessing client-provided reprocessed deep exploration geophysical data. The seismic reflection data to approximately 1250 m BSF are from a line near the Baccalieu Template Sub-site (Plate 1-1.2). The depth of interest for this report appears as the uppermost five horizontal reflections. Note that the base of Unit II appears to undulate around approximately 30 m BSF. This interpretation agrees with data from the Geotechnical Program 2019.

Plate 2.2-2 shows sub-bottom profiler (SBP) data from west to east across the BdNBFA. Vertical resolution is approximately 0.2 m to a penetration depth of approximately 30 m BSF (Fugro, 2019b). This cross-section indicates a relatively high degree of lateral continuity over 16 km. The positions of Units I and II at three geotechnical locations are displayed on the plate. Note that the geotechnical locations displayed on the cross-section are the closest to this transect: they are north of the line at distances indicated on the plate. Nonetheless, due to the relatively high lateral continuity of strata, the occurrences of Units I, II and III on the seismic line and as indicated by the geotechnical data generally coincide.

Plate 2.2-3 shows SBP data from a seismic line 15 m south of geotechnical location EQ19452-BH-51, including a profile of CPT net cone resistance. This cross-section also indicates a relatively high degree

of lateral continuity. The peaks in net cone resistance indicate thin layers of (silty) sand, and possibly occasional gravel and/or cobbles, most of which do not coincide with a seismic reflector.

The geotechnical units described above were defined in this investigation by geotechnical characteristics and relate in a general way to geological stratigraphy. For example, geotechnical Unit I corresponds to extremely low strength Holocene and latest Pleistocene sediments.

2.3 Potential Site-Specific Geohazards

Table 2.1 presents a summary of potential site-specific geohazards.

Table 2.1 is limited to hazards identified using information available at time of the study. A detailed hazard study is not within the scope of this report, and hazards other than those in the table may be present in the site. Locations (i.e. coordinates) of hazards are outside the scope of this report. Refer to other sources, e.g. Fugro (2019b; 2020b) for locations of boulders and fluid escape features, Golder Associates (2014) for seismic hazard information, and Fugro (2017) for information about submarine mass movement deposits.

Table 2.1: Potential Site-specific Geohazards

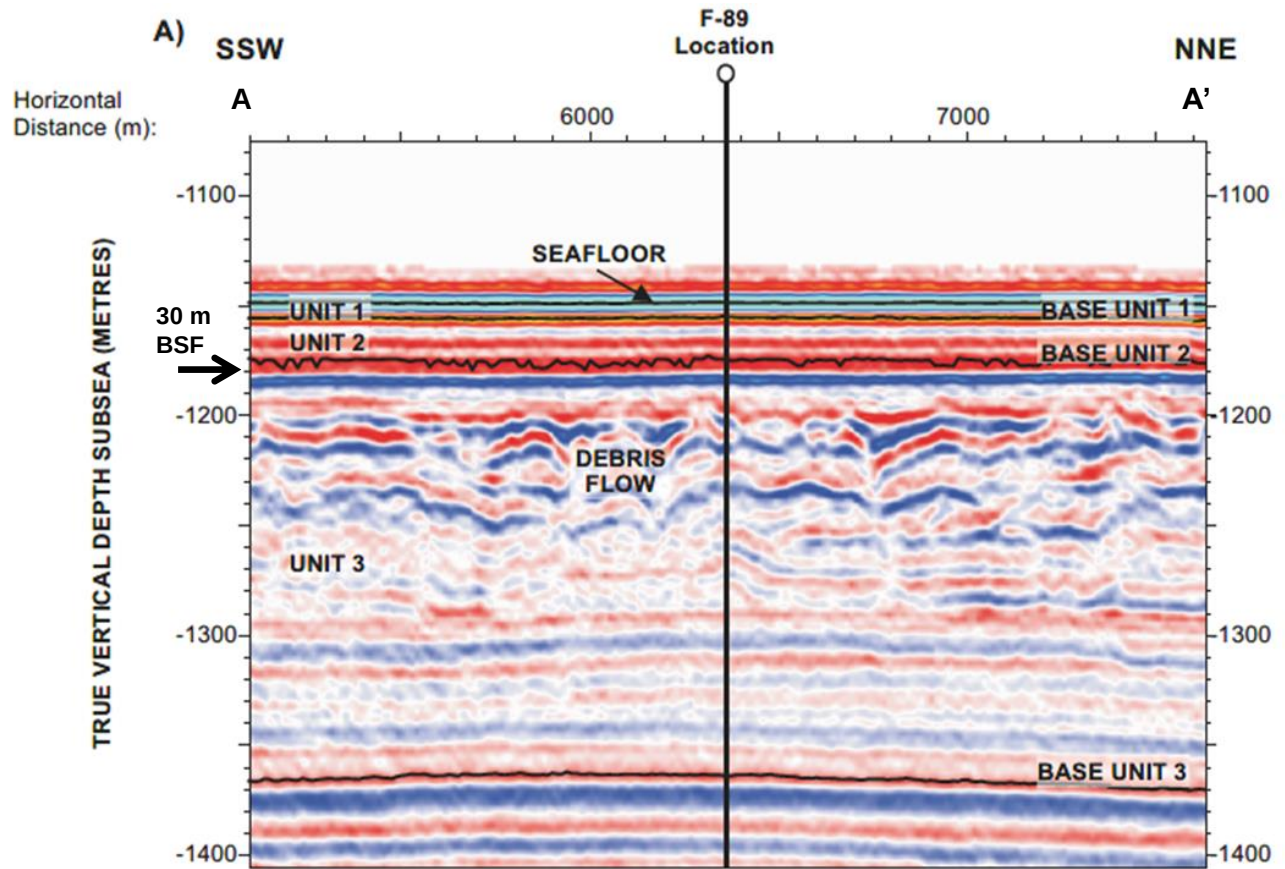
Geohazard	Location / Distance	Possible Impact
Extremely low strength and very low strength material at seafloor	- Present at all borehole locations; assessed as present across the entire site. - Extremely low strength clays at some borehole locations do include gravel or a sand layer	Sliding, uneven penetration of equipment/structures at seafloor
Boulders, cobbles	- Assessed as locally present in the site - At seafloor and in the seabed. - In the Template South Sub-site, including at location EQ19452-BH-21, - In the Template Central Sub-site - In the Template North Sub-site, including at location EQ19452-BH-01 - In the Template Baccalieu Sub-site, including at location EQ19452-BH-31 - No information available from anchor sub-site centred on locations EQ19452-BH-45 and -46. Thus the presence of boulders/cobbles in this sub-site cannot be excluded.	Heterogeneous soil conditions; uneven penetration of equipment/structures; damage to equipment during installation.
Fluid/gas escape features (FEF) (e.g. pockmarks, mounds)	- Assessed as not present in the site, given available information. - No information available from anchor sub-site centred on locations EQ19452-BH-45 and -46. Thus the presence of features in this sub-site cannot be excluded.	Heterogeneous or changing soil conditions. Lower soil strength compared to general soil conditions. Corrosive conditions in soil or water.
Submarine mass movements	In site. Deposits are WLB*	Scour, erosion and soil deposition. Fluid-impact type forces on structure.
Seismicity	Site in Seismic Zone 1 (ISO, 2017).**	Cyclic/ dynamic loading, loss of soil strength.

* Deposits in sub-sites are thin, occasional WLB.

** Assessment of Seismic Zone 1 in ISO 2017 based on 5% dampened spectral acceleration at 1 second oscillation period of 0.1 (g). An update of the standard still in preparation (ISO 2019) indicates Seismic Zone 0, based on damped spectral acceleration value of 0.03 (g).

List of Plates

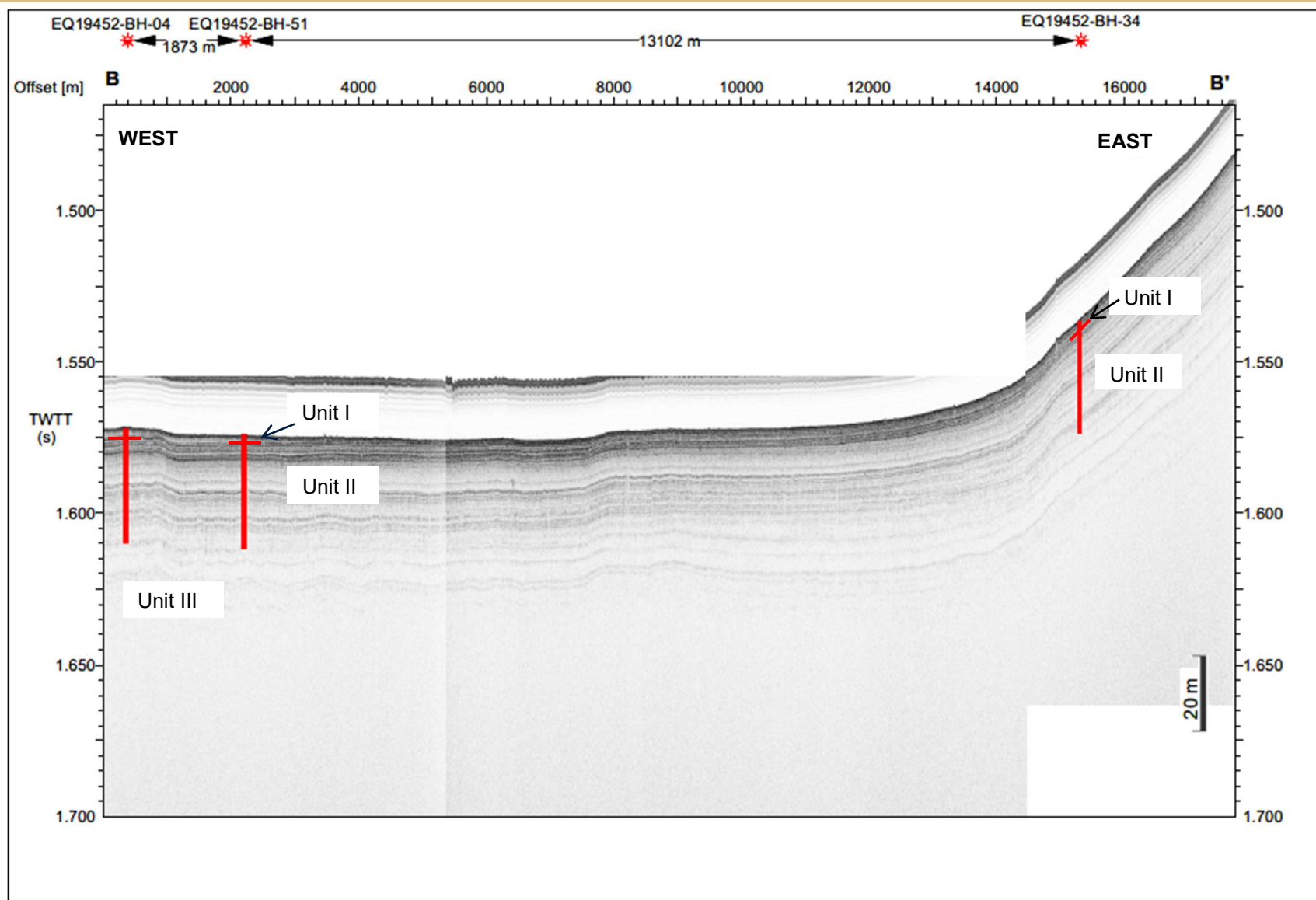
	Plate
Example Regional Stratigraphy	2.2-1 to 2.2-3



Example of 3D seismic cross-section (A-A' on Plate 1.1-2; from Fugro, 2016, Plate 2.5, Figure A, along Inline 835). Units 1, 2, 3 on plate correspond to Units I, II, III in this investigation.

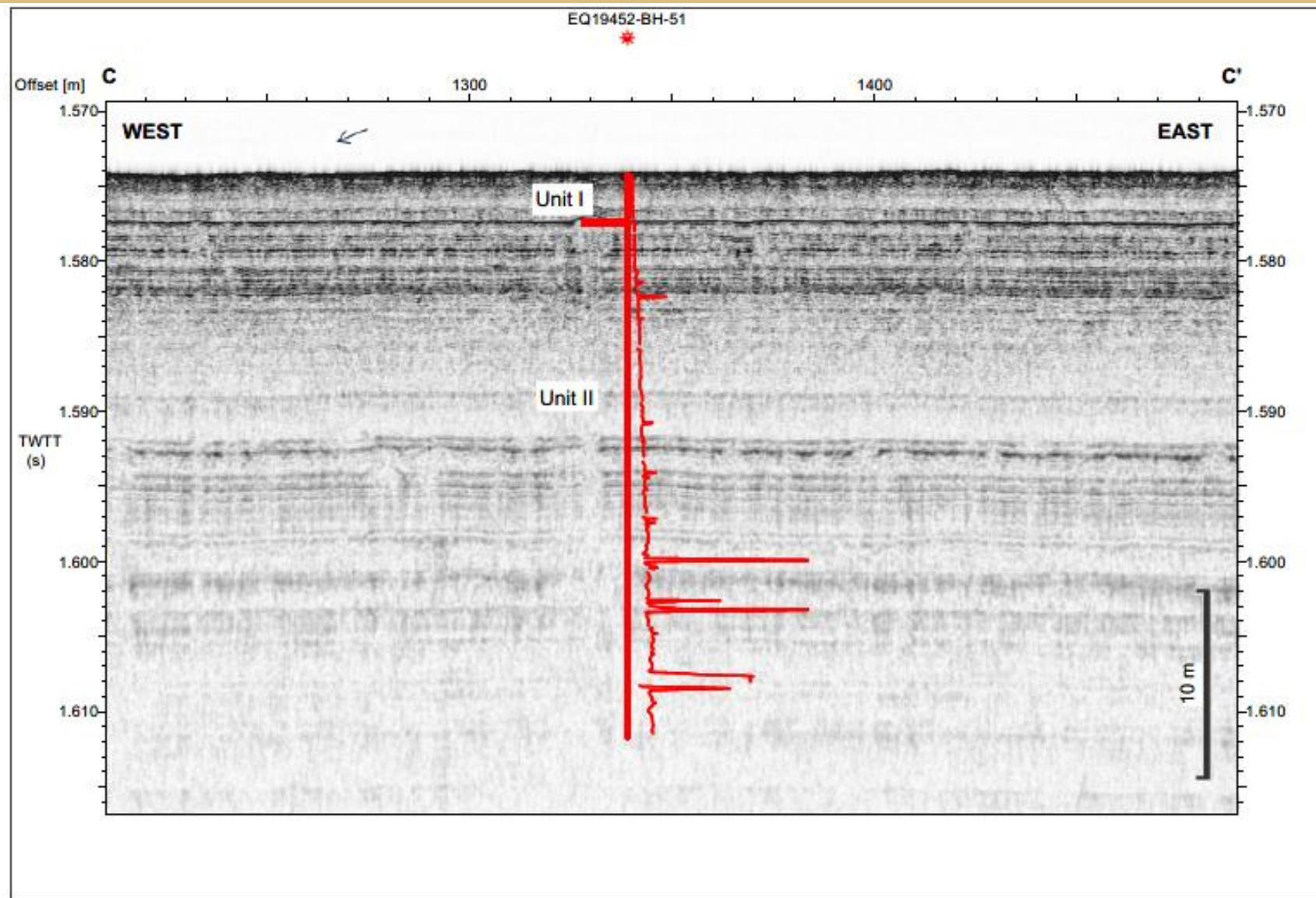
EXAMPLE REGIONAL STRATIGRAPHY NORTH TO SOUTH

BAY DU NORD GP 2019 – ATLANTIC OCEAN, CANADA



SBP cross-section (B-B' on Plate 1.1-2). Borehole locations indicated to 30 m BSF and are north of the line: EQ19452-BH-04, 855 m; BH-51, 490 m; and BH-34, 380 m. Top of Unit III is at approximately 30 m BSF.

EXAMPLE REGIONAL STRATIGRAPHY WEST TO EAST



SBP cross-section (C-C' on Plate 1.1-2). Borehole location EQ19452-BH-51, 15 m north of seismic line, shown with net cone resistance. Top of Unit III is present at approximately 30 m BSF.

EXAMPLE REGIONAL STRATIGRAPHY WEST TO EAST

CHAPTER 3 – EVALUATION OF DATA

3. EVALUATION OF DATA

3.1 Introduction and Use of Results

This chapter provides information on recommended representative geotechnical parameters. The focus for selection of the geotechnical parameters was the usefulness for design of structures such as subsea templates, FPSO anchors, infield flowlines, riser anchors and pipelines. For parameters likely to be required as input for design low, best and/or high estimate profiles are provided.

It is emphasized that the presented derived values can require transformation for appropriate use as representative value or design value in a calculation model (ISO, 2019). Appendix 1 includes guidance, particularly the documents titled “Use of Geodata and Advice” and “Geotechnical Analysis”.

3.1.1 Data Presentation

Table 3.1 shows the geotechnical parameters presented in this report. Plates 3.3-1 to 3.9-9 provide graphical information.

Table 3.1: Recommended Representative Geotechnical Parameters

Geotechnical Parameter	Profile Type	Chapter Sub-section
Net cone resistance (q_n)	BE	3.3.1
Normalized cone penetration test parameters (I_c , Q_{tn} , F_r)	-	3.3.2
Unit weight (γ)	BE	3.4.1
Water content (w)	-	3.4.2
Liquid limit, plastic limit, plasticity index and liquidity index (w_L , w_P , I_P , I_L)	-	3.4.3
Particle diameter (D_n)	-	3.4.4
Thermal conductivity (k)	-	3.4.5
Overconsolidation ratio (OCR)	BE	3.5.1
Undrained shear strength (s_u)	LE, HE	3.6.1 to 3.6.4
Undrained shear strength of remoulded soil ($s_{u,r}$)	LE, HE	3.6.5
Sensitivity (S_t)	BE	3.6.6
Thixotropy strength ratio (T_x)	-	3.6.7
Angle of interface friction (between ground and foundation) (δ)	-	3.7
Constrained modulus (M)	LE, HE	3.8.1
Coefficient of permeability (k)	LE, HE	3.8.2
Coefficient of consolidation (c_v)	LE, HE	3.8.3
External axial strain at half the maximum deviator stress (ε_{50})	BE	3.8.4
Shear modulus at small strain (G_{max})	BE	3.8.5
Normalised shear modulus (G/G_{max})	-	3.9.1
Cyclic undrained strength parameters (τ_{cy} , τ_{av} , N_f)	Drammen Clay ¹	3.9.2
Note(s): Profile type: straight line values for low estimate (LE), high estimate (HE) or best estimate (BE), per soil unit. ¹ Derived values for some of the cyclic parameters are presented within the context of Drammen Clay reference contours (Andersen, 2004).		

The parameter plates show a combined presentation of the locations investigated in the Geotechnical Program 2019 and client-provided geotechnical data (NGI, 2016a; 2016b) where applicable. Data points are plotted according to a colour scheme to identify the different soil units, refer to Chapter 1. Client-provided geotechnical data are presented in grey. A graphic log (soil profile), showing soil unit depth

ranges according to Table 1.2 in the corresponding colour scheme, is provided for the plates where data are plotted versus depth.

In some cases, multiple sets of geotechnical parameter plates are presented for improved visualisation of data and trends. These plates are numbered as. e.g. Plates 3.3-1, 3.3-1A, 3.3-1B. These plates may present:

- i. Filtered CPT data, i.e. peaks in the data, indicative of zones with laminae and beds of silt, sand and/or locally coarser material (WBL zones), were filtered out. Peaks were defined in accordance with outliers, as defined in the section titled “Statistical Analysis”, below. In these cases, client-provided geotechnical CPT data were as omitted from the plates;
- ii. A selection of laboratory test types;
- iii. An enhanced scale from 0.0 m to 3.6 m BSF to better present results for shallow depths.

3.1.2 Low, Best and High Estimate Profiles

The low estimate (LE), best estimate (BE) and high estimate (HE) profiles presented in this report consider the following:

- General soil conditions presented in Chapter 1, typically excluding the possible influence of WBL zones (WBL: with laminae and beds of silt, sand and locally coarser material), unless mentioned otherwise;
- Specific reference of a particular test method or procedure. This is because a derived value depends on the method(s) selected to obtain the parameter value;
- Feasibility of combining derived values from multiple methods in statistical analysis, where applicable and where necessary with application of appropriate:
 - Conversion factors so that values apply to the same reference scale;
 - Weight factors that account for potential bias in data according to a specific method or procedure (DNV GL, 2017);
- Depth trends for inter-dependent geotechnical parameters. As an example, step changes in the BE net cone resistance (q_n) profile at any given depth are also considered for LE and HE undrained shear strength (s_u) profiles at the same depth;
- Judgement and opinion, particularly for a limited quantity or absence of derived values. Judgement and opinion consider physically credible values, comparison of data with results from other tests and a priori knowledge such as geological setting and comparable experience.

Note that the focus is on general soil conditions, i.e. clay. This must be taken into consideration for selection of representative values for design. For example, it can be necessary to consider thin layers of sand within a clay layer for assessing soil resistance for a pipeline at seafloor (DNV GL, 2019).

Appendix 1 includes descriptions of most reference methods considered for this report. Descriptions and details of reference methods are also given in Volume 2 and in the text below, where appropriate.

Some of the derived values of geotechnical parameters include project-specific 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. Inevitably, this introduces uncertainty.

3.1.3 Statistical Analysis

Where possible, LE, BE and HE profiles were calculated using statistical analysis techniques as recommended by DNV GL (2017).

BE profiles were derived based on a corrected mean for each soil unit. The corrected mean was calculated by first establishing the average (x_{av}) and standard deviation (σ) of the dataset, followed by removal of outliers defined as values exceeding the limits of $x_{av} \pm 2\sigma$, followed by recalculation of x_{av} . Note that outliers are not necessarily anomalies. They can represent valid results, for example soil unit weight applicable to a local, thin sand, layer compared to soil unit weight for general soil conditions (clay). Outliers were screened for anomalies, where practicable. Distinct anomalies were excluded from the data considered in this report.

LE and HE profiles aim to represent the quantile associated with the 5 % fractile. LE and HE profiles were evaluated by $x_{av} \pm 2\sigma$ of the corrected dataset, complemented by engineering judgement.

3.2 Data Quality

3.2.1 Geophysical Data

Geophysical data used in this report (Volume 3) appear to meet the requirements of ISO (2018), and include:

- Desk studies about ground in and around the Bay du Nord and Baccalieu Fields (Fugro, 2013; 2014a; 2014b; 2016; 2019c-f) referenced client-provided reprocessed 3D near offset and full offset (3DNO) seismic data. Line spacing was 12.5 m and 25 m. Vertical resolution (limit of separability) of the 3D data ranged from approximately 5 m near the seafloor increasing to 20 m near 2300 m BSF. The desk reports from 2013-2016 stated that the 3D data quality was considered to be high;
- Desk studies also referenced sub-bottom profiler (SBP) and 2D HR seismic data supplied by the Geological Survey of Canada-Atlantic (GSC-A) for large scale conditions to approximately 1250 m BSF.
- A purpose-specific 2018 geophysical investigation: sub-bottom and bathymetry data and a report including assessments of seafloor features and seabed stratigraphy (Fugro, 2019b). Line spacing between the west-east lines was approximately 45 m. There were no crosslines. Penetration of SBP was generally approximately 30 m and vertical resolution of SBP approximately 0.2 m.
- A sub-surface boulder assessment (Fugro, 2020b) based on Fugro, 2019b data;

Reference is made to a Client-provided site-specific seismic hazard analysis (Golder Associates, 2014).

3.2.2 Client-provided Geotechnical Data

The data quality for the client-provided geotechnical data is assessed comparable to that of the Geotechnical Program 2019. Particularly, (1) both data sets were acquired in general agreement with ISO (2014) and (2) the selected methods for in situ testing, sampling, sample handling and laboratory testing were generally similar.

The client-provided geotechnical data (NGI, 2016a; 2016b) apply mainly to locations west of the site, refer to the “Vicinity Map” on Plate 1.1-1. The distance between the closest sub-site and client-provided

geotechnical data is about 800 m, i.e. locations EQ19452-BH-21 and ST15452-CPT-15. Given the regional consistency of the soil conditions, it is viable to consider this data set in this report Volume 3.

3.2.3 Geotechnical Program 2019

The results of in situ tests are of good quality, particularly when considering downhole cone penetration testing and water depths exceeding 1000 m. Zero drift values recorded for the 200 CPTs were within the box values for Application Class 1 of ISO (2014), except for 9 tests which showed zero drift values within the box values of Application Class 2.

Undisturbed sample quality, based on $\Delta e/e_0$ criteria, is generally according to Class 2 or 3 of ISO (2014). This is according to expectations for the selected sampling methods, site conditions and in line with undisturbed sample quality reported for the client-provided geotechnical data. Appendix 1 document titled "Geotechnical Laboratory Tests" includes details on expected effects on laboratory test results.

During the field campaign it was observed that the clay displayed expansive behaviour, forcing itself out of the sampling tube upon retrieval on deck. Occasional, ruptured pores were observed in extruded samples. Such behaviour adversely affects undisturbed sample quality.

3.3 In Situ Profiling

3.3.1 Cone Penetration Test Parameters

Plates 3.3-1, 3.3-1A and 3.3-1B present net cone resistance (q_n) plotted versus depth, including a BE profile.

Client-provided geotechnical data are included on Plates 3.3-1 and 3.3-1B. These data show good consistency with the Geotechnical Program 2019 data.

Plate 3.3-1A presents a plot with peaks filtered out. Plate 3.3-1B presents an enhanced view to a maximum depth of 3.6 m.

The q_n values are consistent for the borehole locations at the site, showing little variation with depth for Unit I and an approximately linear increase for Units II and III, which is typical for normally to slightly overconsolidated clay. The linear increase with depth from seafloor to the considered vertical limit of the site can be expressed as 28 kPa/m.

The q_n intercept at seafloor is typically less than 50 kPa, which is close to the accuracy limits of the CPT results (Peuchen and Terwindt, 2015).

Interpretation of CPT data for Unit I generally indicates extremely low strength clay, occasionally with WBL zones, which can be observed through q_n peaks in the data.

Unit II displays an increasing density of q_n peaks compared to Units I and III. Across the site, at depths varying from 5 m to 8 m BSF, the q_n data indicate the presence of thin beds of gravelly sand, confirmed by the samples retrieved at equivalent depths of Unit II in adjacent boreholes.

Below approximately 20 m BSF to the base of Unit II, the presence of WBL zones increases, and, except for location EQ19452-BH-32, a medium bed of sand is present at depths from 24 m to 30 m BSF.

Unit III indicates a linear increase for q_n , with data peaks indicating WBL zones towards the maximum penetration depth (i.e. 31.8 m BSF).

The presented BE profiles are based on statistical analysis of filtered data for Unit I and Unit II and engineering judgement for Unit III, i.e. focus on the general soil conditions (clay).

The BE profile shows a step increase for q_n at the interface of Unit I and II. This step change corresponds to the difference of these geotechnical units as described in Chapter 2.

It is considered that the BE profile trend estimated for Unit II is also applicable for Unit III.

Consideration was given to statistical analysis focusing on the q_n peaks. Initial results showed a general underprediction of the q_n data. This was attributed to thin-layer effects for cone penetration (Boulanger and DeJong, 2018). No related results are presented in this document.

3.3.2 Normalized Cone Penetration Test Parameters

Soil behaviour type index (I_c), normalized cone resistance (Q_{tn}) and normalized friction ratio (F_r) are presented versus depth on Plate 3.3-2. Comments are generally as for q_n above.

Values for I_c generally range from about 2.9 to 3.2. Based on Robertson and Wride (1998), I_c values ranging between 2.7 and 3.2 classify as “clays – clay to silty clay”, which is in line with visual descriptions and laboratory index test results.

Values of I_c , Q_{tn} and F_r for (and close to) thin sandy layers must be regarded with caution, because of inevitable effects of thin layers on CPT interpretation for individual data points.

3.4 Basic Physical Properties

3.4.1 Soil Unit Weight

Plates 3.4-1 and 3.4-1A present derived values for soil unit weight (γ) versus depth, including a BE profile. Values for γ were obtained using the following methods:

- Volume mass calculated;
- Derived from particle density (ρ) and water content (w).

These methods yielded comparable results.

Results were compared with CPT derived γ , according to Lengkeek et al. (2018) as well as Mayne et al. (2010). Both CPT derived approaches did not fully capture trends and features seen in the results from the above mentioned methods. Significant differences applied to the top 10 metres as well as for Unit III. No γ values are presented derived from CPT correlations, nor considered for deriving the BE profile.

The presented BE profile considers ρ and w derived γ as reference method. Values derived from ρ and w were assessed to be more representative of in situ conditions, considering undisturbed sample quality and observed expansive soil behaviour upon sample extrusion.

BE statistical analysis considered values from both methods in an unweighted manner, as the subsets for the two methods showed comparable mean and standard deviation. Due to the larger quantity of ρ and w derived values, around 70 % of the data, the BE profiles are naturally biased towards γ values of the reference method.

The BE profile for Unit II was defined as a constant value throughout the whole unit. A possible bias towards lower γ values can be noted from 8 m to 13 m BSF. Within this interval q_n displays significantly less peaks and visual sample descriptions suggest the soil as less sandy. However, there are no significant changes in the q_n BE profile, soil strength and soil plasticity for this depth interval.

The BE profile for Unit III is lower than the BE profiles for Unit I and Unit II. The low value may be related to relatively high derived values for liquid limit obtained for samples recovered from Unit III.

3.4.2 Water Content

Plates 3.4-2 and 3.4-2A present water content (w) versus depth.

Comments are as for the section titled “Soil Unit Weight”, above.

No estimate profiles are presented as per Table 3.1.

3.4.3 Soil Plasticity

Plastic limit (w_p) and liquid limit (w_L) and plasticity index versus depth are presented on Plates 3.4-3 and 3.4-4. Plate 3.4-5 presents liquidity index (I_L) versus depth. A plasticity chart is presented on Plate 3.4-6.

The available results are assessed as consistent, with data scatter that is typical for test methods and geological setting.

For the Geotechnical Program 2019, the test method for w_L was the four-point cone penetrometer (fall cone) method. The client-provided geotechnical data are for the one-point cone penetrometer method. The four-point method is typically slightly more reliable.

Calculation of liquidity index requires data pairing. In most cases, same-depth data pairing was done. In case of missing w values, values from nearby tests were used, with specimen depth offsets of up to about 0.1 m. This may have contributed to overall scatter of the I_L results.

The plasticity chart characterises Units I and II as low to high plasticity clay (BSI, 2015). Four of the five data points (Geotechnical Program 2019) for Unit III fall within the soil groups high plasticity silt and very high plasticity silt. Some of these test results may represent layered soil conditions.

3.4.4 Particle Size Distribution

Plate 3.4-7 summarises particle size distribution by means of D_{10} , D_{50} and D_{90} versus depth, where D_n represents particle diameter where n % of the dry mass has a smaller particle diameter.

The laboratory test specimens show percentage of fines (diameter up to 0.06 mm) of typically more than 50 %. The fractions of sand and fine gravel are low.

3.4.5 Thermal Conductivity

Plate 3.4-8 presents derived values for thermal conductivity k versus depth. The values apply to test specimens recovered from Unit I and the upper part (< 3.6 m BSF) of Unit II.

The tests were performed on remoulded specimens, because of operational constraints given the very soft nature of the soil within 4 m BSF.

Note that the test procedure excluded specific measures for ensuring specimen saturation. Partial saturation typically leads to lower values of thermal conductivity compared to saturated conditions.

The presented values fall within the general range given by Farouki (1981). Results are also in line with the client-provided geotechnical data.

3.5 In Situ Stresses and Stress History

3.5.1 Overconsolidation Ratio

Plates 3.5-1 and 3.5-1A present overconsolidation ratio (OCR) versus depth, including a BE profile.

The presented OCR values derived from CPTs are according to Chen and Mayne (1996). The equation is presented in Appendix 1, document titled “Cone Penetration Test Interpretation”.

OCR values derived from oedometer tests generally match with client-provided data. These laboratory values are assessed to be typical for the selected sampling methods, soil conditions and geological setting of the site. Particularly, the relatively low values ($OCR = 1$) derived from the oedometer tests are in line with expectations, considering undisturbed sample quality. It can be noted that undisturbed sample quality class, according to $\Delta e/e_0$ criteria, shows no clear correlation with OCR values obtained from the oedometer tests.

OCR results were derived from anisotropically consolidated undrained triaxial compression (CAU_c) tests, using empirical relationships proposed by Kulhawy (1990) and Andresen (1979). Implausible values, i.e. smaller than 1, were found. No results are presented in this report

The presented BE profile considers the CPT-based method as reference method, determining the general profile shape. The BE profile includes engineering judgement based on laboratory test results. The following comments apply:

- The BE profile for OCR provides a general match with the BE profile for q_n , discussed above;
- OCR results should be used with caution for the upper few metres below seafloor. The CPT-based correlation is then probably at its limits of applicability, considering the low values of q_n ;
- The step-change for q_n at the interface between Unit I and Unit II was omitted for the BE profile for OCR ;
- The general shift between OCR values derived from oedometer tests and CPT-derived values is limited, i.e. a shift from about $OCR = 1.1$ to $OCR = 1.3$ below 13 m BSF.

3.6 Static Undrained Shear Strength

3.6.1 Undrained Shear Strength

Plates 3.6-1 to 3.6-6 present derived values of undrained shear strength (s_u), considering both CPT data and laboratory test data. The presented information includes LE and HE profiles for s_u .

Plates 3.6-1 and 3.6-1A present data pairing results of net cone resistance (q_n) and laboratory s_u . The plates include cone factor N_{kt} , defined here as the ratio of q_n (from BE profiles for Geotechnical Program 2019 data) to s_u derived from laboratory anisotropically consolidated undrained triaxial compression (CAU_c) tests ($N_{kt} = q_n/s_{uCAUc} = 12$). The selected value of N_{kt} was assessed by methods of linear approximation (Plate 3.6-1A). The CAU_c data considered the q_{max} failure criterion for s_u , where q_{max} is the maximum value of principal deviator stress within 0 to 15 % axial strain. The N_{kt} value was also checked against the q/p'_{max} failure criterion, where q represents principal deviator stress and p' represents mean effective stress. The q/p'_{max} approach would have given $N_{kt} = 15$. Additional information on the N_{kt} method can be found in Appendix 1, document titled “Cone Penetration Test Interpretation”.

Plate 3.6-1 includes values of $q_n/s_{uDSS} = 17$ and $q_n/s_{uCAUe} = 20$ for s_u values derived from laboratory direct simple shear (DSS) tests and anisotropically consolidated undrained triaxial extension (CAU_e) tests. Section 3.6.2 includes further discussion.

Plate 3.6-2 presents data pairing results of excess pore pressure ($\Delta u = u_2 - u_0$) derived from BE CPT data and laboratory s_u derived from CAU_c tests considering the q_{max} failure criterion for s_u . The presented results include a trend line for pore pressure factor $N_{\Delta u} = \Delta u/s_{uCAUc}$. The value $N_{\Delta u} = 7.5$ suggested in the client-provided geotechnical data is in line with the data points from the Geotechnical Program 2019.

Plate 3.6-3 presents N_{kt} values versus depth. The presented data points are the same as shown on Plate 3.6-1A. The N_{kt} values show no significant trend with depth. In order to increase confidence in the selected $N_{kt}=12$ value, additional analysis was performed using a CPT pore pressure ratio (B_q) approach (Plate 3.6-4), according to the method proposed by Mayne and Peuchen (2018) which can be found in Appendix 1, document titled “Cone Penetration Test Interpretation”. Good consistency was observed between the linear approximation method and the B_q method.

Plate 3.6-5 presents $N_{\Delta u}$ values versus depth. The presented data points are the same as shown on Plate 3.6-2. The $N_{\Delta u}$ values show no significant trend with depth, as for N_{kt} . It can be noted here that N_{kt} shows smaller relative scatter (standard deviation over selected value) than $N_{\Delta u}$. N_{kt} was thus selected as preferred correlation factor.

Plates 3.6-6, 3.6-6A and 3.6-6B present undrained shear strength (s_u) versus depth, including LE and HE profiles. Plate 3.6-6 presents an overview, including derived values from selected laboratory test methods and client-provided geotechnical data. Plate 3.6-6A presents filtered data and Plate 3.6-6B presents data to a maximum depth of 3.6 m BSF.

Plates 3.6-1, 3.6-6 and 3.6-6A include s_u values derived from CAU_c , DSS and CAU_e tests. The results of these test types for the Geotechnical Program 2019 show good agreement with the client-provided geotechnical data.

Values of s_u acquired offshore using pocket penetrometer (PP), torvane (TV), laboratory vane (LV), fall cone (FC) and unconsolidated undrained triaxial (UU) compression test methods are assessed to be within expected ranges/scatter for the type of soils and sampling/test methods. Index strength values of s_u that are less than about 5 kPa should be regarded with caution, as such low values typically approach the applicability limits for the sampling and test methods. Low values typically apply to test specimens recovered from Unit I. The general trend of s_u values derived from the index strength tests is lower than that for values of s_u derived from onshore CAU_c and direct simple shear (DSS) tests.

The LE and HE profiles consider CPT-based s_u values obtained with $N_{kt} = 12$.

The results of the Geotechnical Program 2019 were prioritized for assessment of the LE and HE profiles, i.e. the distance to the client-provided geotechnical data was considered.

The LE and HE profiles for Unit I are based on statistical analysis of the CPT derived data and consider engineering judgement to account for the (very) low values of q_n (Peuchen and Westgate, 2018). The low values approach the applicability limit of the cone penetration test method. No project-specific CAU_c data are available for Unit I. Any CAU_c data would have been outside the applicability range for this test method. The provided s_u LE profile is comparable with the s_u representative low profile suggested for the soil unit at seafloor in client-provided data.

The LE and HE profiles show a step change in s_u for the interface between Unit I and Unit II. This matches the step change for the corresponding BE profile for q_n .

The LE and HE profiles for Unit II, between 2 m and 7.8 m BSF, consider engineering judgement, particularly to allow for slightly overconsolidated conditions and to provide a link with the LE and HE profiles for Unit II below 7.8 m BSF. The LE profile considers a projected intercept of $s_u = 0.5$ kPa at seafloor and s_u increasing at 4 kPa/m depth BSF. The HE profile considers a projected intercept of $s_u = 10$ kPa at seafloor, which is the same value as for the HE profile for Unit I. The linear increase for the HE profile is the same as for the LE profile.

The LE and HE profiles for Unit II below 7.8 m BSF and for Unit III consider the BE profile for q_n and N_{kt} values of 10 and 14. Note, these values are merely used to obtain depth dependent LE and HE profiles for s_u and do not represent low or high estimates for N_{kt} itself. The selected N_{kt} range captures most of the s_u values derived from the CAU_c tests and implicitly considers possible combinations of low and high values of q_n and N_{kt} .

3.6.2 Strength Anisotropy

Comparison of s_u trends for strength anisotropy indicate anisotropy ratios of about 0.7 for DSS and 0.6 for CAU_e results, where s_u derived from CAU_c results (q_{max} failure criterion) serves as denominator, refer to Plate 3.6-1. These anisotropy ratios are in line with common ratios reported by Andersen et al. (2008).

Note that strength anisotropy, as used in this report section, is based on different modes of shearing of soil, for which different values of undrained shear strength would be expected. This difference in undrained shear strength can be related to factors such as test method, test boundary conditions and orientation of critical shear planes. Note that the presented results consider a default orientation of test specimens, e.g. no DSS tests were conducted on test specimens at 90° orientation with respect to in situ conditions.

3.6.3 Remoulded Undrained Shear Strength

Plates 3.6-7, 3.6-7A and 3.6-7B present remoulded undrained shear strength ($s_{u,r}$) values, including LE and HE profiles. Plate 3.6-7 presents an overview, including in situ test data and all the $s_{u,r}$ values derived from laboratory test results. Plate 3.6-7A combines $s_{u,r}$ derived from the fall cone (FC) results derived from in situ test data. Data presented on Plate 3.6-7B duplicate those of Plate 3.6-7, to a maximum depth of 3.6 m BSF.

The laboratory test results presented on the plates can be grouped as follows:

- Laboratory index strength tests on remoulded test specimens, i.e. the FC, unconsolidated undrained triaxial compression (UU) and direct simple shear (DSS) methods. None of the index strength methods include specific procedures for ensuring specimen saturation;
- Laboratory index strength test on intact test specimen, whereby remoulding is achieved by large displacement, i.e. the laboratory vane (LV) method;
- Correlation with CPT sleeve friction f_s according to Ramsey (2010), i.e. $s_{u,r} = 0.66 \cdot f_s$.

The results for the Geotechnical Program 2019 and the client-provided geotechnical data show reasonable agreement.

Values of $s_{u,r}$ that are less than about 5 kPa should be regarded with caution, as such low values typically approach the applicability limits for the sampling and test methods. Low values frequently apply to test specimens recovered from Unit I and from Unit II above about 20 m BSF.

The general range of test results and data scatter is according to expectations for the soil conditions and the different test methods. Data scatter possibly increases for zones with a higher frequency of WBL, i.e. may be influenced by test premises for soil homogeneity and specimen saturation.

The CPT-based method shows limited data scatter compared to FC results and a clear depth trend for $s_{u,r}$. This depth trend shows reasonable correspondence with that for the FC results.

DSS results present $s_{u,r}$ values that are significantly higher than $s_{u,r}$ derived from any other tests. Checks for water content, unit weight and shear strain showed no evidence for errors and outliers. The DSS results are presented together with the other test results on Plate 3.6-7. It is recommended to consider the DSS results as anomalies.

The general trend of results was checked against correlations based on liquidity Index (I_L) following a statistical procedure based on Skempton and Northey (1952).

The presented LE and HE profiles consider the FC method as reference method. Comments are as follows:

- The profiles are based on geostatistics for the FC results and consider the depth trend for $s_{u,r}$ for the CPT-based method;
- Considerations for the LE profile include engineering judgement, i.e. a seafloor intercept of $s_{u,r} = 0.125$ kPa and an increase of $s_{u,r}$ equivalent to 0.25 kPa per metre depth BSF (as lower limits). A LE profile based solely on FC results would fall in line or even be lower;
- The seafloor intercept of $s_{u,r} = 2.5$ kPa for the HE profile is approximate and considers the applicability limits for the FC method;
- A few FC results are available for Unit III. These results appear to fall within the general LE and HE profiles for Unit II.

3.6.4 Strength Sensitivity

Plates 3.6-8 and 3.6-8A present sensitivity (S_t) values versus depth, including a BE profile.

The presented S_t values consider data pairing per (laboratory) index strength method for $s_{u,r}$ described above, i.e. $s_u/s_{u,r}$ for the FC, UU, DSS and LV methods. The presented comments for s_u and $s_{u,r}$ also apply to strength sensitivity. The uncertainties associated with data pairing are additional.

The results for the Geotechnical Program 2019 and the client-provided geotechnical data show reasonable agreement.

There is significant spread in the results, with S_t values typically ranging from 1 to 10, and single values exceeding 10. The spread in results is according to expectations.

In line with the remoulded undrained strength LE and HE profiles, the presented S_t BE profile considers data pairing of FC test results as reference method. FC tests results present lower scatter than LV, even though both tests generally follow the same trend and fall in line with the BE profile (Plate 3.6-8). Also, LV remoulded testing, unlike FC, was not performed on pre-treated samples, i.e. results could be influenced by sand laminae or other discontinuities.

The BE profile approaches a statistical fit for the FC results and includes engineering judgment considering the inevitable uncertainty of the input values.

3.6.5 Thixotropy

Plates 3.6-9 and 3.6-10 present laboratory thixotropy test results, expressed as thixotropy strength ratio $T_x(t) = s_{u;ar}(t)/s_{u;r}$, where t refers to time, $s_{u;ar}$ represents undrained shear strength of aged remoulded soil and $s_{u;r}$ represents undrained shear strength of remoulded soil at $t = 0$.

The laboratory test results apply to clay soil recovered from Unit II.

Plate 3.6-9 includes a comparison of derived values of T_x with literature values (Andersen, 2008) and (the inverse of) clay activity A , i.e. the ratio of plasticity index to percentage by weight of clay-size particles. Comments are as follows.

- Andersen (2008) indicated a possible weak trend between T_x and $1/A$. The data scatter for the available test results is similar to that presented by Andersen (2008) and provides no conclusive evidence for contradicting or confirming this weak trend;
- Presented values for $1/A$ were derived from the nearest Atterberg limits and particle size distribution results in the same borehole. In case results were available immediately above and below the thixotropy test, both results were plotted, as for example was done for client-provided geotechnical data;
- Witthoeft et al. (2017) noted that clays with a plasticity index between 30 % and 50 % have highest thixotropic potential, and clays with a plasticity index of < 30 % show least potential;
- Witthoeft et al. (2017) shows that < 50 % plasticity index clays show no indications of stabilizing within 2 month testing periods available;
- Some results indicate $T_x < 1$. This is attributed to uncertainty of derived values of $s_{u;ar}$ and $s_{u;r}$. These results were considered for calculation of ranges and for calculation of mean values of T_x .
- Results generally indicate a rapid strength increase after 1 day. The strength keeps increasing up to 60 days, though it is noted that with elapsing time, more scatter is introduced;
- Results of the Geotechnical Program 2019 fall within general scatter of literature values, though it is noted that results deviate somewhat from the (approximate) relation with plasticity index presented by Witthoeft et al. (2017);
- Clays in the Bay du Nord and Baccalieu fields are characterised by a plasticity index of ~20 %, and by a relatively low clay content, refer to Sections titled “Soil Plasticity” and “Particle Size Distribution” respectively. Clay content considered is as low 10 %, largely attributed to high fraction of silt-sized particles. Standard deviation of clay contents considered is significantly larger than the rather constant observed plasticity index. This is to be considered when comparing results to literature values.

Plate 3.6-10 presents minimum, maximum and mean observed thixotropic ratio for the Geotechnical Programme 2019.

3.7 Angle of Interface Friction

Table 3.2 summarises results of three soil-steel interface ring shear tests performed on specimens from Unit II clays, at approximately 6 m, 15 m and 20 m BSF. The presented results are for a test phase aimed at fully drained conditions.

Table 3.2: Angle of Interface Friction

Unit	Angle of Interface Friction (δ) [°]	
	Peak	Residual
I	No Data	No Data
II	33.6	29.7
	27.0	23.8
	33.0	29.4
III	No Data	No Data

The presented derived values are in line with δ values for clay with a relatively low plasticity index (Jardine, 2005).

3.8 Stiffness, Permeability and Volume Change Behaviour

3.8.1 Constrained Modulus

Plate 3.8-1, Plate 3.8-1A and Fig. 3.1 present derived values for constrained modulus M at an effective vertical stress that approximates in situ effective vertical stress. Plates 3.8-1 and 3.8-1A include LE and HE profiles. Plate 3.8-1A shows filtered data.

The test results presented on the plates can be grouped as follows:

- Results from incremental loading (IL) and constant rate of strain (CRS) oedometer tests, considering $M = 1/m_v$ where m_v represents coefficient of volume compressibility;
- Correlation with laboratory undrained shear strength s_u derived from CAU_c triaxial compression, UU triaxial compression and DSS tests;
- Correlation with CPT net cone resistance q_n according to Kulhawy and Mayne (1990).

The oedometer test methods include no specific procedures for ensuring specimen saturation.

The CRS test results typically showed pore pressure ratios of less than 3 % for the target effective vertical stress. The 3 % value is a lower limit recommended by ASTM (2012) and ISO (2014). Checks with high rates of strain (i.e. above the 10 %/h recommended by ISO for high plasticity silts), pore pressure ratios of > 3 % were not achieved. This setting reduces the reliability of the CRS-derived values for M .

Correlation of s_u and M is similar to correlation of q_n and M , except that the latter typically provides more consistent results. The data points for the CAU_c tests consider $M/s_u = 85$. For the CAU_c derived values (q_{max} failure criterion), this ratio provides a reasonable fit with the oedometer test results (Fig. 3). Note that the presented data fitting included removal of some outliers.

The UU triaxial compression test and the DSS test are index strength tests, with no specific control of effective stress conditions during shearing. The presented M values consider the M/s_u ratio selected for the CAU_c test results, thus resulting in generally lower M values.

The data points for the q_n correlation consider $M = 7 \cdot q_n$. The factor 7 provides a good fit to the derived values for the IL test method and is slightly lower than the factor 8.25 suggested by Kulhawy and Mayne (1990).

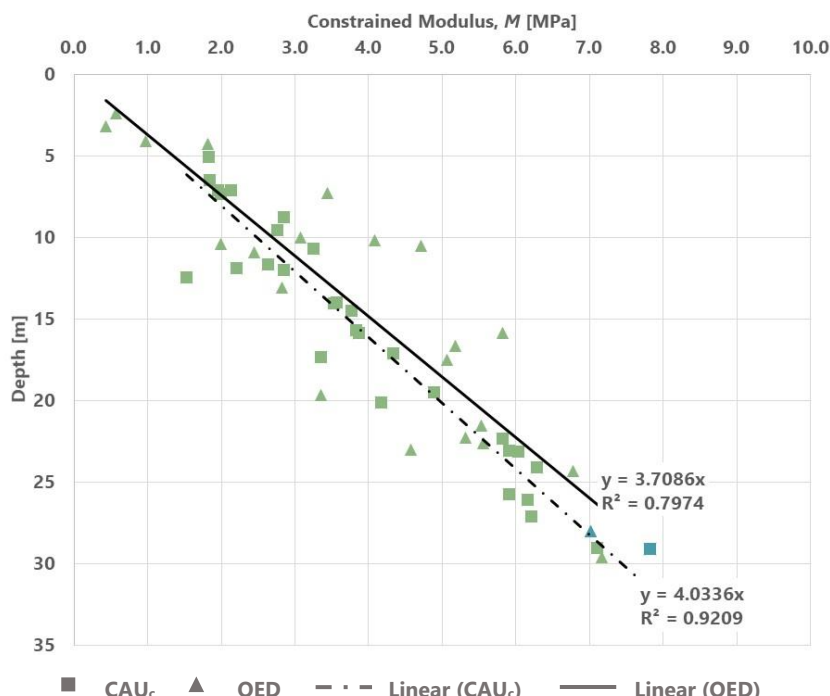


Figure 3.1: M from CAU_c and oedometer tests versus depth

The presented LE and HE profiles consider the IL oedometer method as reference method.

The trend lines follow the general shape of the BE line for q_n and the LE and HE profiles for s_u described above.

3.8.2 Coefficient of Permeability

Plate 3.8-2 presents derived values for permeability k , including LE and HE profiles.

The test results presented on the plates cover three test methods:

- Constant head test in triaxial cell, including specimen saturation and application of effective stress conditions that are approximately equal to in situ stress conditions;
- Incremental loading (IL) and constant rate of strain (CRS) oedometer tests.

No derived values are available for Unit I.

The k values derived from the constant head permeability tests are generally slightly lower than the k values derived from the oedometer tests.

Comments on the oedometer test method are generally as for constrained modulus, above. The k values derived from CRS test results for the client-provided geotechnical data were calculated at zero axial strain.

A reasonable agreement can be observed between the client-provided geotechnical data and the data from the Geotechnical Program 2019.

The presented LE and HE profiles consider the constant head permeability test as reference method.

The LE and HE profiles are based on engineering judgement. Comments are as follows:

- Consideration was given to the derived values from the oedometer tests, as the derived values for the oedometer methods broadly agree with those from the constant head permeability tests;
- The extension of the LE and HE profiles to cover Unit I is based on engineering judgement, particularly the general correspondence of Unit I soils with those of Unit II;
- The few data points available for Unit III agree with derived values for Unit II, as expected.

3.8.3 Coefficient of Consolidation

Plate 3.8-3 presents derived values for coefficient of consolidation c_v at an effective vertical stress that approximates in situ effective vertical stress. The plate also includes LE and HE profiles.

The presented values apply to laboratory oedometer tests results, particularly incremental loading (IL) and constant rate of strain (CRS) test types. Comments on the oedometer test methods are as discussed for constrained modulus, above.

Five data points are available for the IL method, all of which apply to test specimens from Unit II. The larger number of data points for the CRS method cover test specimens recovered from Units II and III. No specific laboratory test data are available for Unit I.

The derived values for the CRS method broadly match those for the IL method.

The derived values for the Geotechnical Program 2019 show good agreement with the client-provided geotechnical data.

The presented LE and HE profiles consider the IL oedometer method as reference method.

The LE and HE profiles are based on engineering judgement. Comments are as follows:

- Consideration was given to the derived values from CRS tests, as the derived values for the CRS method broadly agree with the IL values;
- The extension of the LE and HE profiles to cover Unit I is based on engineering judgement, particularly the general correspondence of Unit I soils with those of Unit II;
- The few data points available for Unit III agree with derived values for Unit II, as expected.

3.8.4 External Axial Strain at Half the Maximum Deviator Stress

Plate 3.8-4 presents axial stiffness represented by derived values for axial strain at half the maximum deviator stress (ϵ_{50}), including a BE profile.

The presented values were derived from laboratory UU and CAU_c triaxial compression test results available for Units II and III. No derived values are available for Unit I. No UU-based values are available to Unit III.

The ϵ_{50} values derived from the UU tests are generally higher than those derived from the CAU_c tests, which is according to expectations for the differences between these two test methods.

The Geotechnical Program 2019 covered both test types. Among the client-provided data no ϵ_{50} values are available.

The presented BE profile considers the UU triaxial compression method as reference method. This choice is according to general use of ϵ_{50} values, e.g. empirical calculation methods for lateral pile design described by ISO (2016).

The BE profile for Unit II is based on geo-statistics. Values for ϵ_{50} exceeding 3 % were considered as outliers and not used for geo-statistics. The BE profile for Unit I is based on engineering judgement, particularly taking account of the occurrence of extremely low strength soils. The BE profile for Unit II was extended to Unit III.

3.8.5 Small Strain Behaviour

Plate 3.8-5 presents derived values for shear modulus at small strain (G_{max}), including a BE profile.

The test results presented on the plate cover:

- Correlation with CPT parameters according to Robertson and Cabal (2015);
- Results from laboratory resonant column tests;
- Results from bender elements tests on laboratory triaxial test specimens.

The primary input parameter values for the CPT based method are net cone resistance q_n and soil behaviour type index I_c . The derived parameter is shear wave velocity v_s . Values for G_{max} were derived from $G_{max} = \rho \cdot v_s^2$, where ρ represents density of ground. Background information is presented in Appendix 1, document titled “Cone Penetration Test Interpretation”.

Laboratory procedures for resonant column tests and bender elements tests included recompression of intact test specimens to approximate in situ stress conditions and included specimen saturation. The derived values from the laboratory tests mainly apply to specimens recovered from Unit II. No laboratory test data are available for Unit I. The laboratory values generally agree with those from the CPT correlation.

The presented BE profile considers the CPT-based correlation as reference method, as this provides continuous data through all three soil units.

The general shape of the BE profile for G_{max} is according to the BE profile for q_n , described above, including a step change at the interface between Unit I and Unit II. The selected approach is thus based on engineering judgement and implicitly incorporates geo-statistics by means of the BE profile for q_n .

The derived values for G_{max} for the Geotechnical Program 2019 generally match with the client-provided geotechnical data.

3.9 Cyclic and Dynamic Soil Behaviour

3.9.1 Shear Modulus Degradation and Viscous Damping

3.9.1.1 Normalised Shear Modulus

Plate 3.9-1 presents derived values of G/G_{max} versus cyclic shear strain amplitude γ_{cy} , where G is shear modulus and G_{max} is shear modulus at small strain. The presented results are for 6 laboratory test specimens from Unit II and Unit III, respectively. The data apply to the resonant column method and cover shear strain values of $< 0.1\%$.

The client-provided geotechnical data exclude resonant column test results. No project-specific values for G/G_{max} are available for Unit I.

The majority of test results shows good agreement with published research at representative ranges of plasticity index I_p (Vucetic and Dobry, 1991; Vardanega and Bolton, 2013) indicated on the plates. Results for 2 tests (RC03 and RC05) give G/G_{max} values outside of the expected range. The test results for RC03 and RC05 were specifically inspected for characteristics such as specimen unit weight and Atterberg limits. The inspection indicated no evidence for anomalies.

3.9.1.2 Damping Ratio

Plate 3.9-2 presents derived values of (viscous) damping ratio λ versus cyclic shear strain amplitude γ_{cy} . The presented results are for the resonant column tests discussed in the sub-chapter “normalised shear modulus”, above.

The majority of test results shows good agreement with published research at representative ranges of plasticity index I_p (Vucetic and Dobry, 1991) indicated on the plates. Results for one test (RC03) give λ values outside of the expected range. Comments are as for “Normalised Shear Modulus”, above.

3.9.2 Cyclic Shear Strength

3.9.2.1 Normalised Cyclic Shear Stress – CSS – 0.1 Hz

Plate 3.9-3 presents derived values of τ_{cy}/τ_{ref} and τ_{av}/τ_{ref} with respect to N_f (number of cycles to soil failure or final number of cycles), where τ_{cy} represents cyclic shear stress amplitude, τ_{ref} represents static shear stress as reference and τ_{av} represents average shear stress. The plate includes a site-specific contour model incorporating both client-provided geotechnical data and the Geotechnical Program 2019 data. For comparison, the plate also includes a reference Drammen Clay model (Andersen, 2004) for an equivalent $OCR = 1$ and a contour model presented as part of the client-provided data. Note that the presented values for $N_f = 1500$ apply to N_f as final number of cycles at test termination, rather than N_f as number of cycles to soil failure.

Plate 3.9-4 presents derived values of τ_{cy}/τ_{ref} for selected values of γ_{cy} (0.5 %, 1 %, 5 %, 10 % and 15 %). Plates 3.9-5 to 3.9-6 presents derived values of τ_{cy}/τ_{ref} for selected values of γ_{av} (0.5 %, 1 %, 5 %, 10 % and 15 %). The presented derived values are plotted versus number of cycles N and grouped according to normalised average shear stress τ_{av}/τ_{ref} . The plates also include S-N trend lines, with S-

N representing abbreviations for τ_{cy}/τ_{ref} and N . For comparison, these plates include reference S-N curves for the Drammen Clay model for an equivalent $OCR = 1$. It should be noted that the site-specific S-N curves are based on a simple power law. This method, although likely to be cautious for low cyclic stress ratios for high numbers of cycles, is considered satisfactory.

The values of τ_{cy} , τ_{av} , N and N_f were obtained from cyclic direct simple shear (CSS) tests on intact test specimens obtained from Unit II. No project-specific data are available for Units I and III. As for DSS, the CSS test method excludes specific test procedures for ensuring specimen saturation. The presented CSS data are for stress-controlled conditions and a cycling frequency of 0.1 Hz ($T = 10$ s). This frequency is commonly applied for input in assessment of wave loading conditions.

Values for τ_{ref} are equal to derived values of undrained shear strength s_u obtained from static DSS test results for nearby test specimens (Table 3.3). In other words, τ_{cy}/τ_{ref} and τ_{av}/τ_{ref} imply data pairing and associated uncertainties.

The results for the Geotechnical Program 2019 and the client-provided geotechnical data show reasonable agreement.

The site-specific model in 3D (τ_{av} , τ_{cy} , N_f) interaction space provides a reasonable fit to the reference Drammen Clay model (Plate 3.9-3). Key data points for the site-specific model match with the site-specific S-N curves (Plates 3.9-5 to 3.9-6). Note that derivation of a 3D framework for prediction of γ_{av} and γ_{cy} is beyond the scope of this report. If such a framework is required for design, the Drammen clay model (Andersen, 2004) is expected to provide a cautious representation of Unit II and Unit III data, based on the comparisons presented in this report.

Based on the applied τ_{av} value for a given test, test specimens will fail under a combination of γ_{cy} and γ_{av} . The general observation is a failure mode occurring under γ_{av} for 1-way loading with τ_{av} equal to or in excess of 20 % of τ_{ref} . Tests performed under 2-way loading ($\tau_{av} = 0$) indicate a failure mode under γ_{cy} . These observations conform reasonably well with those made with the Drammen Clay model (Andersen, 2004); however, it was observed that samples were subject to failure under accumulation of γ_{av} at lower τ_{av} values than inferred from the Drammen clay model.

Table 3.3 presents the selected tests used for derivation of the CSS S-N trend lines. The table includes derived values for plasticity index I_p applicable to nearby test specimens and (static) undrained strength ratio s_u/σ'_{vo} , where σ'_{vo} is effective in situ vertical stress. Tests are presented according to a colour scheme, refer to Chapter 1. The client-provided geotechnical data are presented in grey.

Table 3.4 presents curve fitting coefficients for the presented S-N curves, using Equation 3.9.1. Note that the S-N curves exclude a 3D (τ_{av} , τ_{cy} , N) interaction space.

$$\frac{\tau_{cy}}{\tau_{ref}} = a \cdot N^b \quad \text{Equation 3.9.1}$$

Table 3.3: Test Grouping for S-N Curves (CSS, 0.1 Hz, Stress-controlled)

$\tau_{av}/\tau_{ref}=0.0$				$\tau_{av}/\tau_{ref}=0.2$				$\tau_{av}/\tau_{ref}=0.4 \text{ to } 0.5$			
Ref. Test	Test ID [-]	s_u/σ'_{vo} [-]	I_P [%]	Ref. Test	Test ID [-]	s_u/σ'_{vo} [-]	I_P [%]	Ref. Test	Test ID [-]	s_u/σ'_{vo} [-]	I_P [%]
DSS18	CSS22	0.25	19	DSS22	CSS04	0.25	18	DSS01	CSS01	0.35	14
	CSS23				CSS05				CSS02		
	CSS24				CSS06				CSS03		
DSS08	CSS10	0.24	18	DSS19	CSS19	0.28	20	DSS16	CSS16	0.25	25
	CSS11				CSS20				CSS17		
	CSS12				CSS21				CSS18		
·	8B2DSSCy	0.27	17					·	8D5DSSCy	0.27	17
	8B3DSSCy										
	8B4DSSCy										

Table 3.4: Curve Fitting Coefficients for S-N Curves (CSS, 0.1 Hz, Stress-controlled)

γ_{cy} or γ_{av} [%]	N [-]	Curve Fitting Coefficients					
		$\tau_{av}/\tau_{ref}=0.0$		$\tau_{av}/\tau_{ref}=0.2$		$\tau_{av}/\tau_{ref}=0.4 \text{ to } 0.5$	
		a	b	a	b	a	b
0.5	1 to 10 000	0.93	-0.108	0.84	-0.110	0.58	-0.090
1	1 to 10 000	1.05	-0.119	1.01	-0.128	0.85	-0.128
5	1 to 10 000	1.29	-0.139	1.19	-0.142	1.00	-0.133
10	1 to 10 000	1.35	-0.142	1.29	-0.146	1.10	-0.140
15	1 to 10 000	1.37	-0.142	1.31	-0.142	1.15	-0.142
Note(s): a and b = curve fitting coefficients Refer to text for selection of strain mode (γ_{cy} or γ_{av})							

3.9.2.2 Normalised Cyclic Shear Stress – CTXL – 0.1 Hz

As described for CSS above, Plate 3.9-7 presents derived values of τ_{cy}/τ_{ref} and τ_{av}/τ_{ref} with respect to N_f , including a site-specific contour model, a reference Drammen Clay model (Andersen, 2004) for an equivalent $OCR = 1$, and a contour model presented as part of the client-provided data. Note that the presented values for $N_f = 1500$ and $N_f = 5000$ apply to N_f as final number of cycles at test termination, rather than N_f as number of cycles to soil failure.

Plates 3.9-8 and 3.9-9 present derived values of τ_{cy}/τ_{ref} versus N , for selected values values of γ_{av} (0.25 %, 1 %, 5 %, 10 % and 15 %) and grouping according to τ_{av}/τ_{ref} , as described for CSS above. The plates also include S-N trend lines. For comparison, both plates include reference S-N curves for the Drammen Clay model for an equivalent $OCR = 1$. It should be noted that the site-specific S-N curves are based on a simple power law. This method, although likely to be cautious for low cyclic stress ratios for high numbers of cycles, is considered satisfactory.

The presented values of τ_{cy} , τ_{av} , N and N_f were obtained from cyclic consolidated undrained triaxial (CTXL) tests on intact test specimens obtained from Unit II and Unit III. No project-specific data are available for Unit I. The presented CTXL data are for stress-controlled conditions and a cycling frequency of 0.1 Hz ($T = 10$ s).

Values for τ_{ref} are equal to derived values of s_u obtained from static anisotropically consolidated undrained triaxial compression (CAU_c) test results for nearby test specimens (Table 3.5), using q_{max} as failure criterion for s_u , where q_{max} is the maximum value of principal deviator stress within 0 to 15 % axial strain. Values of τ_{cy}/τ_{ref} and τ_{av}/τ_{ref} imply data pairing and associated uncertainties.

The results for the Geotechnical Program 2019 and the client-provided geotechnical data show reasonable agreement.

The site-specific model in 3D (τ_{av} , τ_{cy} , N_f) interaction space indicates a trend of slightly higher values of τ_{cy}/τ_{ref} compared to the reference Drammen Clay (Plate 3.9-7). Key data points for the site-specific model match with the site-specific S-N curves. Note that derivation of a 3D framework for prediction of γ_{av} and γ_{cy} is beyond the scope of this report. If such a framework is required for design, the Drammen clay model (Andersen, 2004) is expected to provide a cautious representation of Unit II and Unit III data, based on the comparisons presented in this report.

Note that some of CTXL tests forming part of the Geotechnical Program 2019 ($\tau_{av}/\tau_{ref} = 0.3$) could not be taken to beyond approximately 10 % average shear strain (γ_{av}) before failing. The results of these tests could not be used for deriving the corresponding S-N contours for γ_{av} of 10 % and 15%.

Based on the applied τ_{av} value for a given test, test specimens will fail under a combination of γ_{cy} and γ_{av} . The general observation for the test results of the Geotechnical Program 2019 (τ_{av}/τ_{ref} of > 0.25) is a failure mode occurring under γ_{av} . This is to a degree similar to the Drammen Clay model (Andersen, 2004), which indicates failure under τ_{av}/τ_{ref} of > 0.4 .

Table 3.5 presents the selected tests used for derivation of the CTXL S-N trend lines. Comments are as for the CSS results described above. **Table 3.6** presents curve fitting coefficients for the presented S-N curves, as for the CSS results described above.

Table 3.5: Cyclic Test Grouping for S-N Curves (CTXL, 0.1 Hz, Stress-controlled)

$\tau_{av}/\tau_{ref} = 0.3$				$\tau_{av}/\tau_{ref} = 0.5 \text{ to } 0.6$			
Ref. Test	Test ID [-]	s_u/σ'_{v0} [-]	I_p [%]	Ref. Test	Test ID [-]	s_u/σ'_{v0} [-]	I_p [%]
CAUcBE35	CTXL19	0.60	19	CAUcBE36	CTXL01	0.36	19
	CTXL21				CTXL02		
CAUc23	CTXL13	0.40	21	CAUc12	CTXL04	0.44	27
CAUc27	CTXL14	0.40	19		CTXL05		
-	6B2CAUcy	0.37	15	CAUc23	CTXL12	0.37	21
	6B3CAUcy			CAUc29	CTXL16	0.38	20
				CAUcBE35	CTXL20	0.60	19

Table 3.6: Curve Fitting Coefficients for S-N Curves (CTXL, 0.1 Hz, Stress-controlled)

γ_{av} [%]	N [-]	Curve Fitting Coefficients			
		$\tau_{av}/\tau_{ref} = 0.3$		$\tau_{av}/\tau_{ref} = 0.5 \text{ to } 0.6$	
		a	b	a	b
0.25	1 to 10 000	0.55	-0.087	0.55	-0.113
1.0	1 to 10 000	0.65	-0.10	0.65	-0.110
5.0	1 to 10 000	0.81	-0.12	0.72	-0.110
10.0	1 to 10 000	0.82	-0.12	0.76	-0.120
15.0	1 to 10 000	0.83	-0.12	0.78	-0.115
Note(s): a and b = curve fitting coefficients					

3.9.3 Post-cyclic Undrained Shear Strength – DSS and CSS – 0.5 Hz

Figure 3.2 presents derived values of post-cyclic (static) undrained shear strength s_{uPcy} by normalisation, particularly s_{uPcy}/τ_{ref} , where τ_{ref} represents static shear stress as reference. The values of s_{uPcy}/τ_{ref} are plotted versus τ_{cy}/τ_{ref} , where τ_{cy} represents cyclic shear stress amplitude.

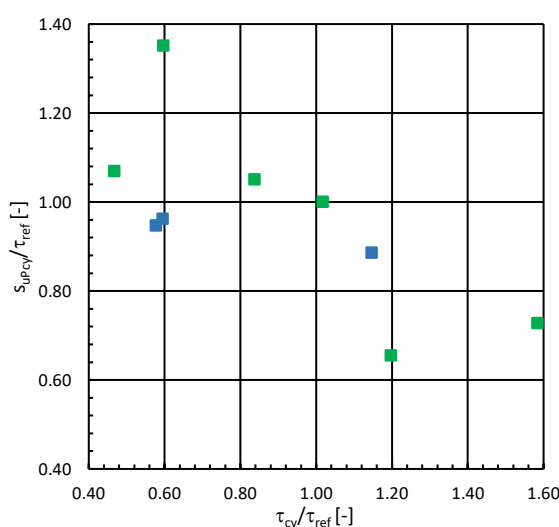


Figure 3.2: Normalised post cyclic and monotonic strength ratio versus cyclic stress ratio

The laboratory values of s_{uPcy} were obtained from a direct simple shear (DSS) test phase, performed immediately after a cyclic direct simple shear (CSS) test phase on the same soil specimen. The strain rate for the DSS test phase was about 3 %/h, i.e. as per usual DSS strain rates.

Values for τ_{ref} are equal to derived values of s_u obtained from static DSS test results for nearby test specimens (Table 3.6). In other words, s_{uPcy}/τ_{ref} , τ_{cy}/τ_{ref} and τ_{av}/τ_{ref} imply data pairing and associated uncertainties.

The CSS test phase comprised stress-controlled cycling at a frequency of 0.5 Hz ($T = 2$ s). Tests were performed at τ_{av}/τ_{ref} ratios of 0 to 0.2, where τ_{av} is average shear stress. The criterion for termination of the cyclic test phase was $N = 10$, where N is cycle number. The cycling frequency (0.5 Hz) and the selected N criterion are commonly applied for input in assessment of seismic (earthquake) loading conditions.

Table 3.7 presents a data summary. Note that the target criterion $N = 10$ at $T = 2$ s was largely met.

The DSS and CSS tests were performed on intact test specimens obtained from Units II and III. No project-specific data are available for Unit I. No comparable client-provided geotechnical data are available. The DSS and CSS test methods exclude specific test procedures for ensuring specimen saturation.

The limited test data suggest a possible trend of no significant post-cyclic strength reductions for $\tau_{cy}/\tau_{ref} < 1$ and for combinations of $\gamma_{cy} < 15$ % and $\gamma_{av} < 15$ % at N .

Table 3.7: Summary of Post-Cyclic Strength Results

Test ID	τ_{ref}	s_{uPcy}	N	τ_{cy}/τ_{ref}	τ_{av}/τ_{ref}	γ_{cy}	γ_{av}
[-]	[kPa]	[kPa]	[-]	[-]	[-]	[%]	[%]
DSS21	53	-	-	-	-	-	-
CSS13		57	10	0.56	0.19	0.19	0.38
CSS14		72	10	0.68	0.21	0.63	0.36
CSS28		56	10	0.75	0.19	13.0	14.3
DSS13	42	-	-	-	-	-	-
CSS07		28	10	0.84	0.00	2.30	0.07
CSS08		18	9	1.08	0.00	15.0	-1.75
CSS09		20	3	1.33	0.00	15.0	-0.19
DSS23	65	-	-	-	-	-	-
CSS25		63	10	0.56	0.00	0.32	-0.02
CSS26		67	10	0.76	0.00	1.07	-0.04
CSS27		64	10	0.93	0.00	1.57	0.08
Note(s): Values of γ_{cy} and γ_{av} apply to cycle number N							

3.9.4 Strain Rate Dependency for Cyclic Strength

Figure 3.3 presents derived values of τ/τ_{ref} versus rate of (shear) strain ($\dot{\gamma}$), obtained from the static DSS tests presented in Table 3.8. The parameter τ represents static shear stress at a selected rate shear strain between 3 %/hour and 12000 %/hour. The parameter τ_{ref} represents τ for a reference rate of (shear) strain $\dot{\gamma}_{ref} = 3$ %/hour, in line with current practice for DSS testing. Figure 3.3 includes a trend line according to Equation 3.9.2, for coefficient $m = 0.06$.

$$\frac{\tau}{\tau_{ref}} = \left(\frac{\dot{\gamma}}{\dot{\gamma}_{ref}} \right)^m$$

Equation 3.9.2

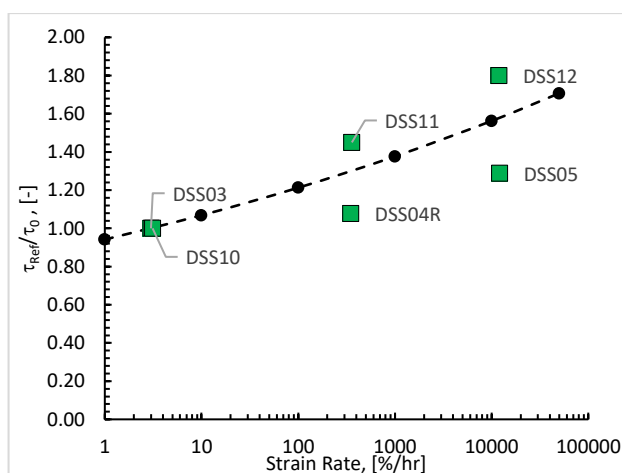


Figure 3.3: Normalised (static) shear stress versus rate of strain

Values for τ (and τ_{ref}) are equal to derived values of s_u obtained from the static DSS test results (Table 3.8).

The values of τ/τ_{ref} imply data pairing for $\dot{\gamma} > \dot{\gamma}_{ref}$. The uncertainties associated with data pairing are additional to those of a single DSS test.

The DSS tests were performed on intact test specimens obtained from Unit II. No project-specific data are available for Units I and III. No comparable client-provided geotechnical data are available. The DSS test method excludes specific test procedures for ensuring specimen saturation.

Table 3.8: Summary of Strain Dependent DSS Results

Test ID	Depth, z	Rate of Strain, $\dot{\gamma}$	Undrained Shear Strength, s_u (τ)	Normalised Undrained Strength, τ/τ_{ref}	Shear Strain at Failure, γ	Shear Induced Pore Pressure, Δu	Intact Sample Quality Index, $\Delta e/e_0$
[-]	[m]	[%/hr]	[kPa]	[-]	[%]	[kPa]	[-]
DSS03	20.60	3.1	52	1.00	15.0	69	0.105
DSS04R	20.65	350	56	1.08	15.0	60	0.129
DSS05	20.70	12088	67	1.29	15.0	24	0.115
DSS10	10.60	3.1	20	1.00	6.7	29	0.085
DSS11	10.65	359	29	1.45	7.4	27	0.058
DSS12	10.70	11943	36	1.80	9.7	26	0.056
Note(s): Δu represents a calculated value, applicable to shear strain at failure γ $\Delta e/e_0$ represent an intact (undisturbed) sample quality index (ISO, 2014), described in Appendix 1							

The coefficient $m = 0.6$ of Equation 3.9.2 is typical for laboratory DSS test results for normally consolidated clays (Rattley et al., 2011). The trend line indicates an approximate 16 % increase in s_u per log cycle increase in $\dot{\gamma}$.

The trend line of Equation 3.9.2 can be used for more in-depth interpretation of laboratory cyclic strength, for example by applying Equation 3.9.3:

$$\tau_{ref}^* = \tau_{ref} \cdot \left(\frac{R_m}{R_p} \right) \quad \text{Equation 3.9.3}$$

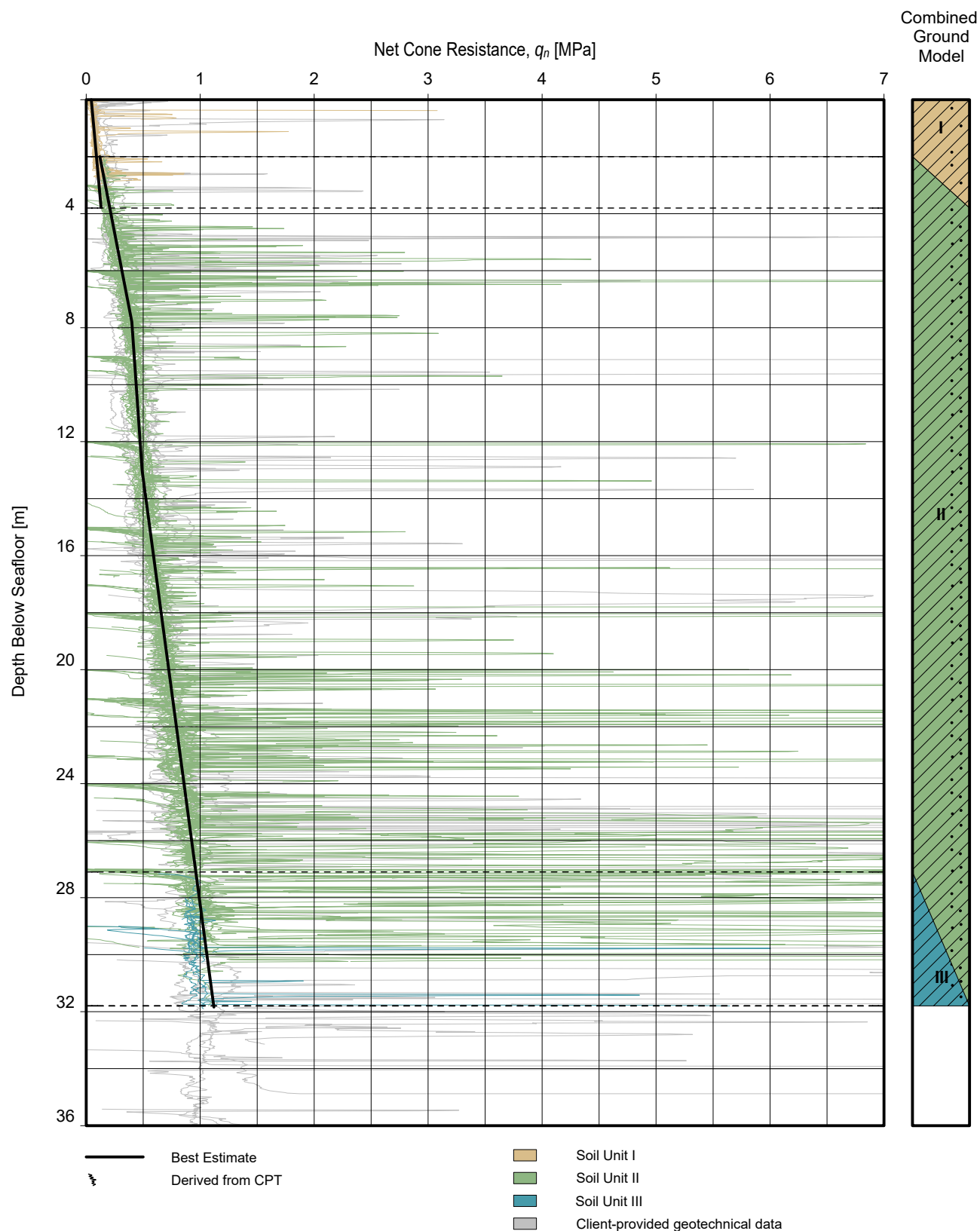
where τ_{ref}^* represents a corrected τ relative to τ_{ref} . The parameters R_m and R_p are rate dependency parameters that can be illustrated by the following example:

- Estimate the average rate of strain for the first quarter cycle of a CSS by considering the maximum shear strain γ_{max} and cyclic frequency of the test;
- Compare the derived value of shear stress τ at each derived value of γ in the first quarter cycle of the CSS with τ at the same value of γ in the respective static DSS, and derive R_m for the paired test results;
- Calculate τ_{ref}^* from Equation 3.9.3, taking R_m from Step ii and taking $R_p = \tau/\tau_{ref}$ from Equation 3.9.2.

Rate dependency is typically independent of the mode of shearing (Graham et al., 1983). This suggests that adjustment of τ_{ref} to τ_{ref}^* can also applied to the cyclic triaxial test results, using DSS results.

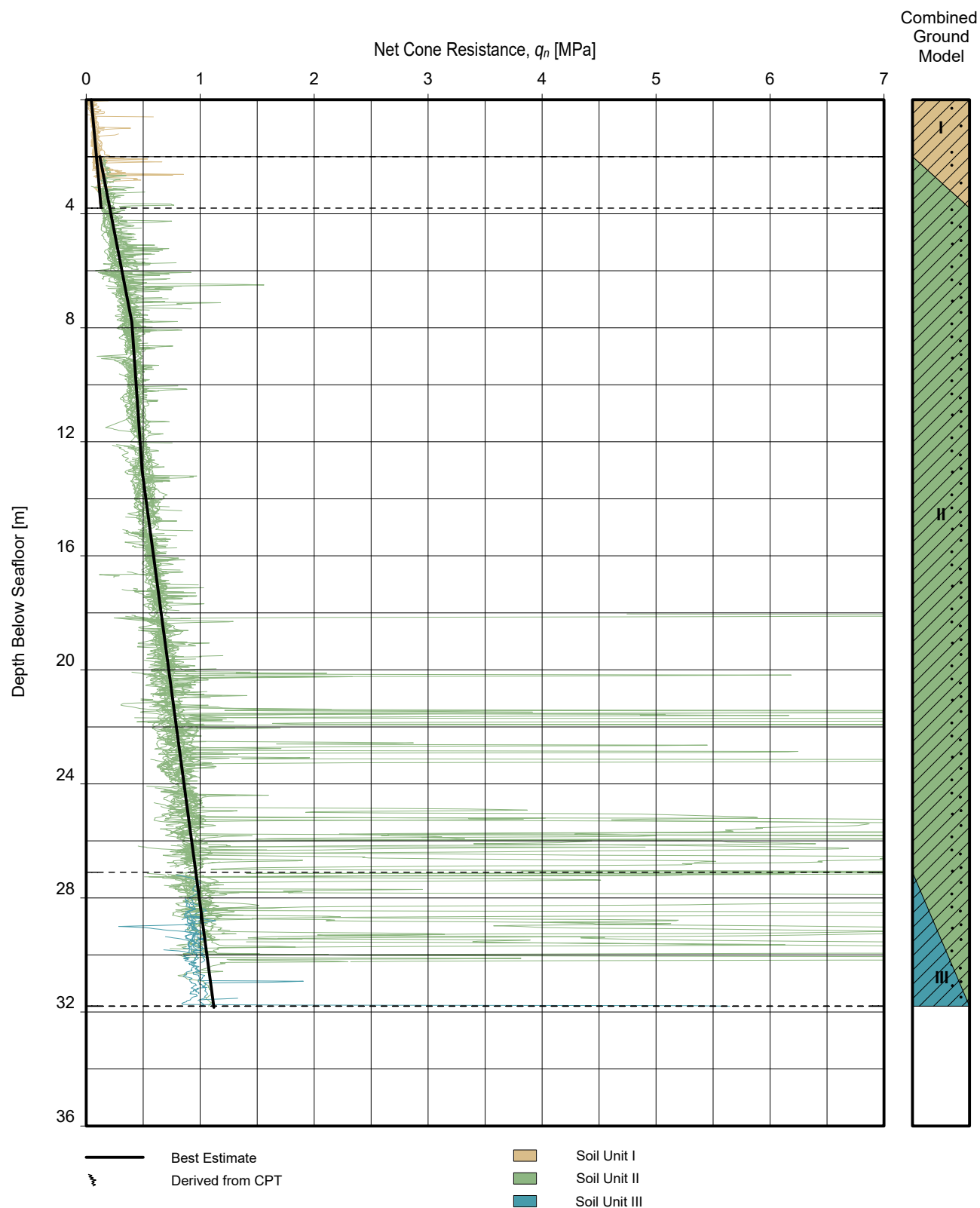
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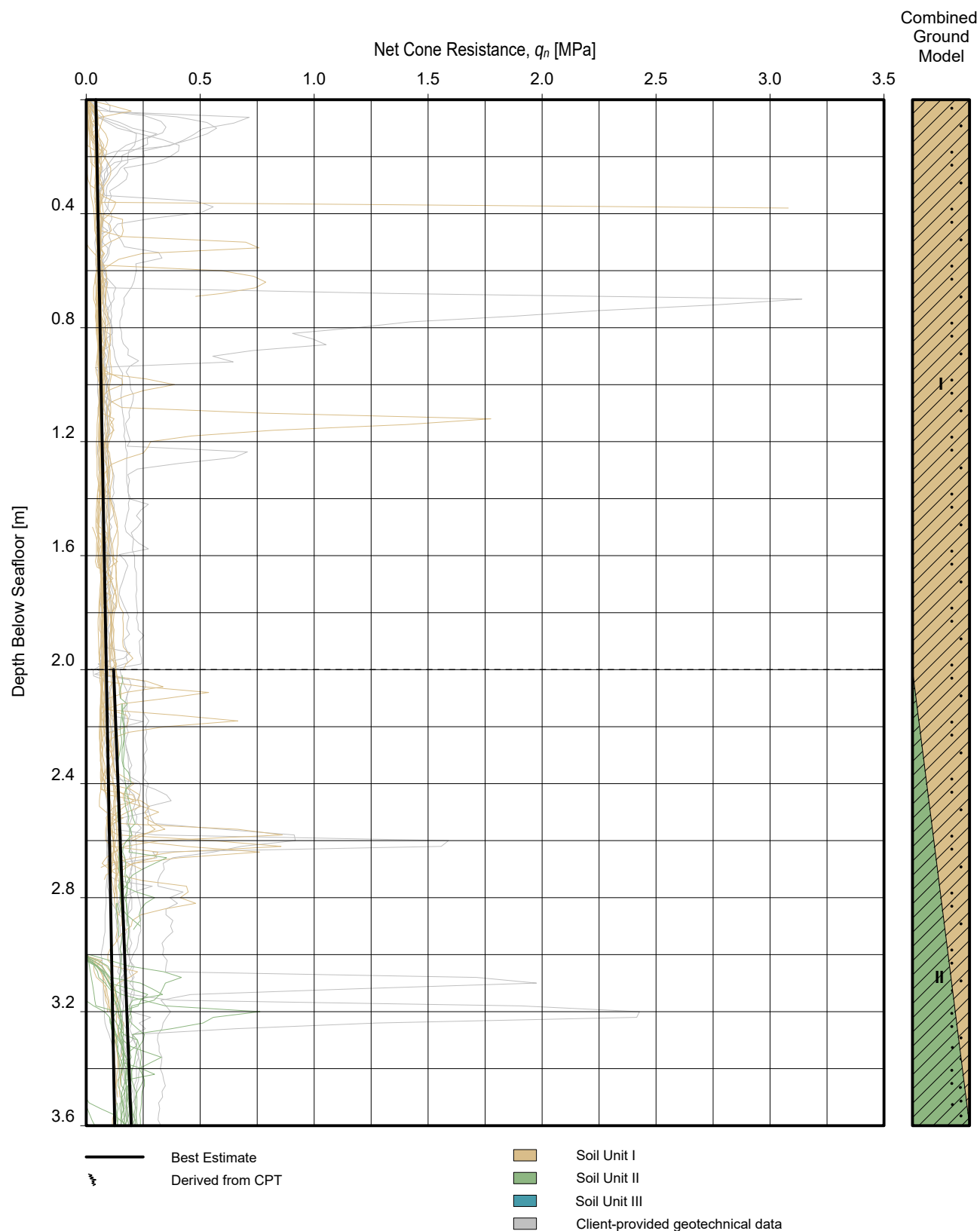
Note(s):
 - Refer to Chapter 3 for more information about units and locations represented

NET CONE RESISTANCE VERSUS DEPTH



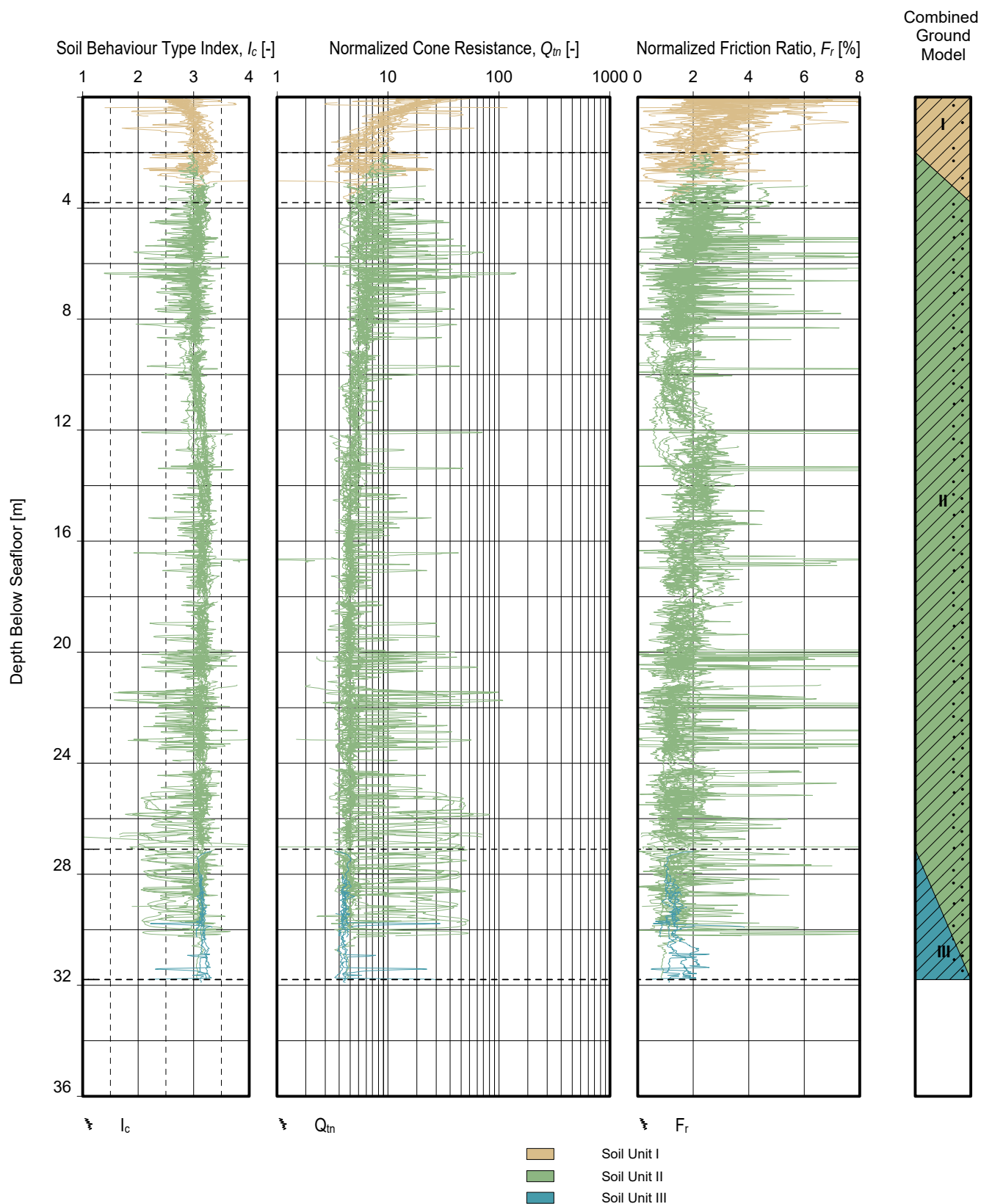
Note(s):
 - Refer to Chapter 3 for more information about units and locations represented

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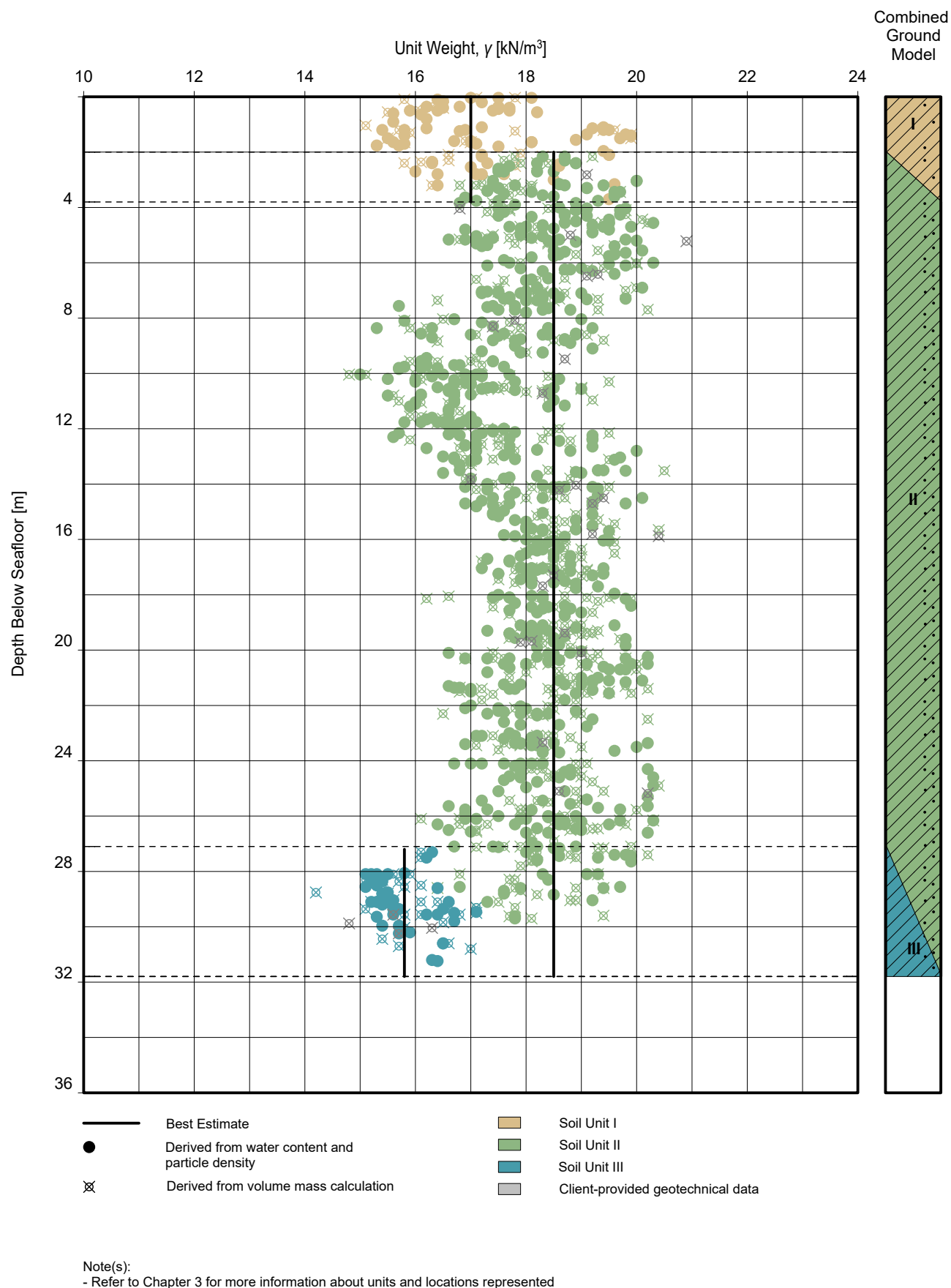
Note(s):
- Refer to Chapter 3 for more information about units and locations represented

NET CONE RESISTANCE VERSUS DEPTH



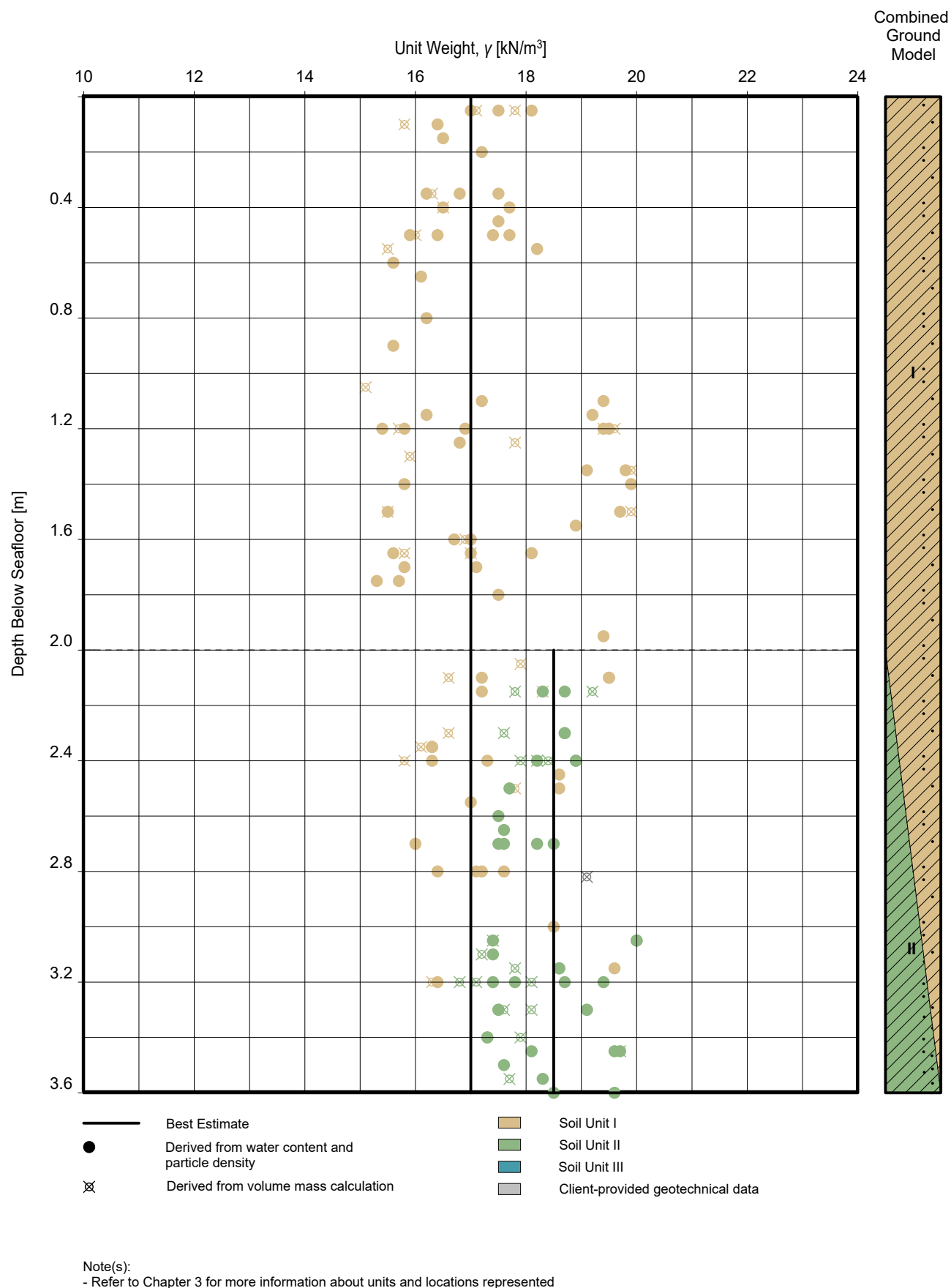
Note(s):
- Refer to Chapter 3 for more information about units and locations represented

NORMALIZED CPT PARAMETERS VERSUS DEPTH

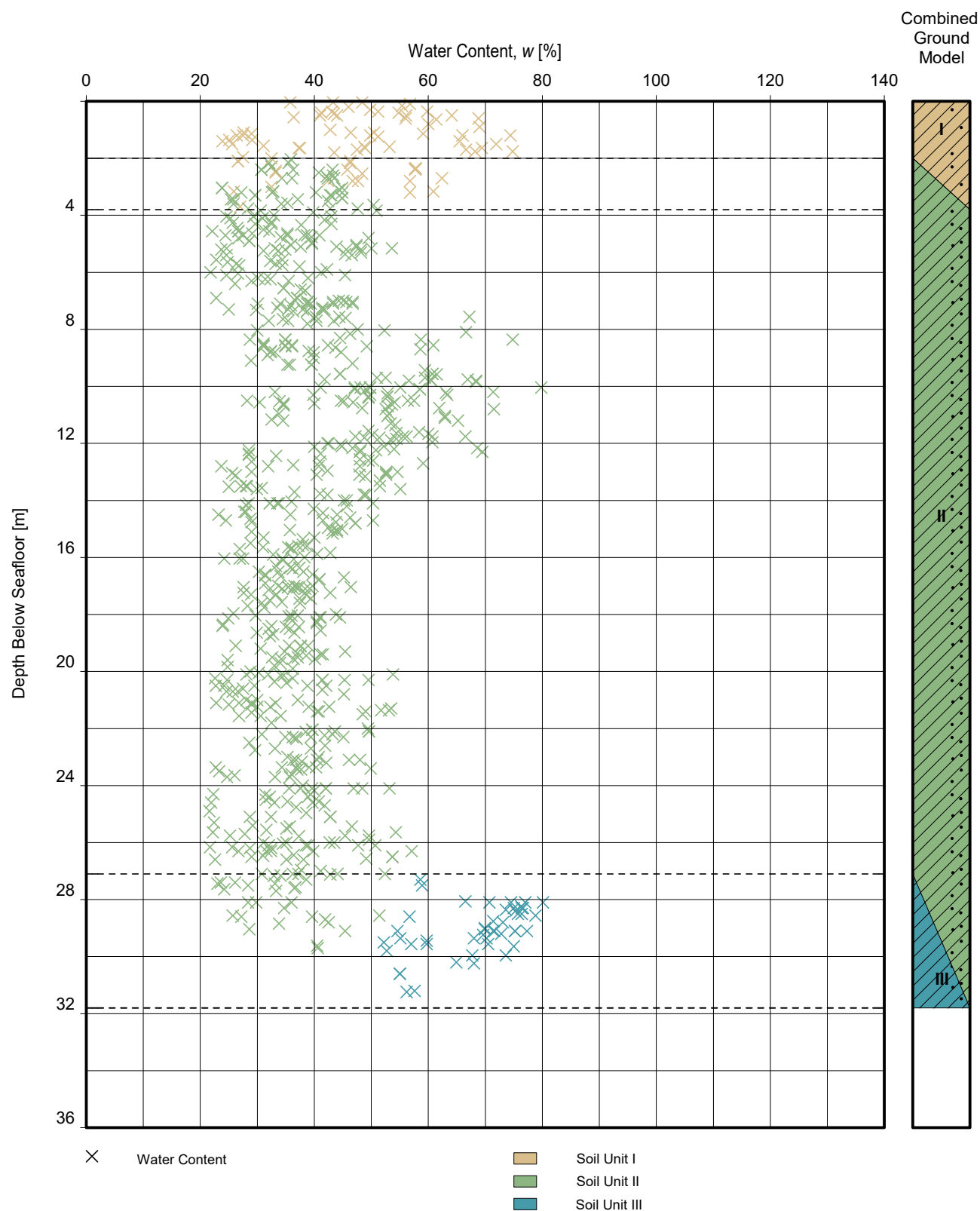


GeODir/3.4-2a_Unit Weight vs Depth - No CPT_36.0 m.GGF/2020-03-06 10:14:25

UNIT WEIGHT VERSUS DEPTH

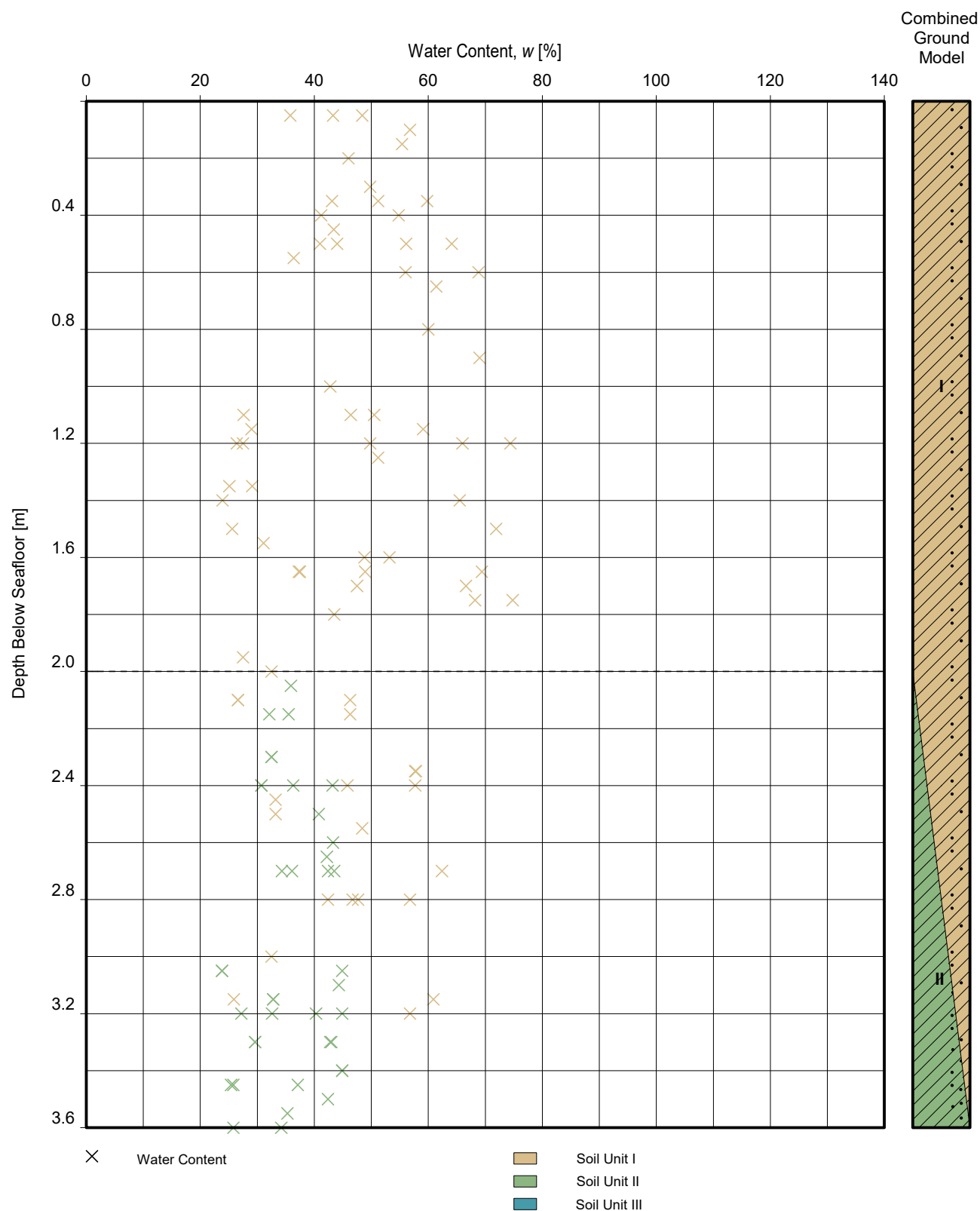


UNIT WEIGHT VERSUS DEPTH



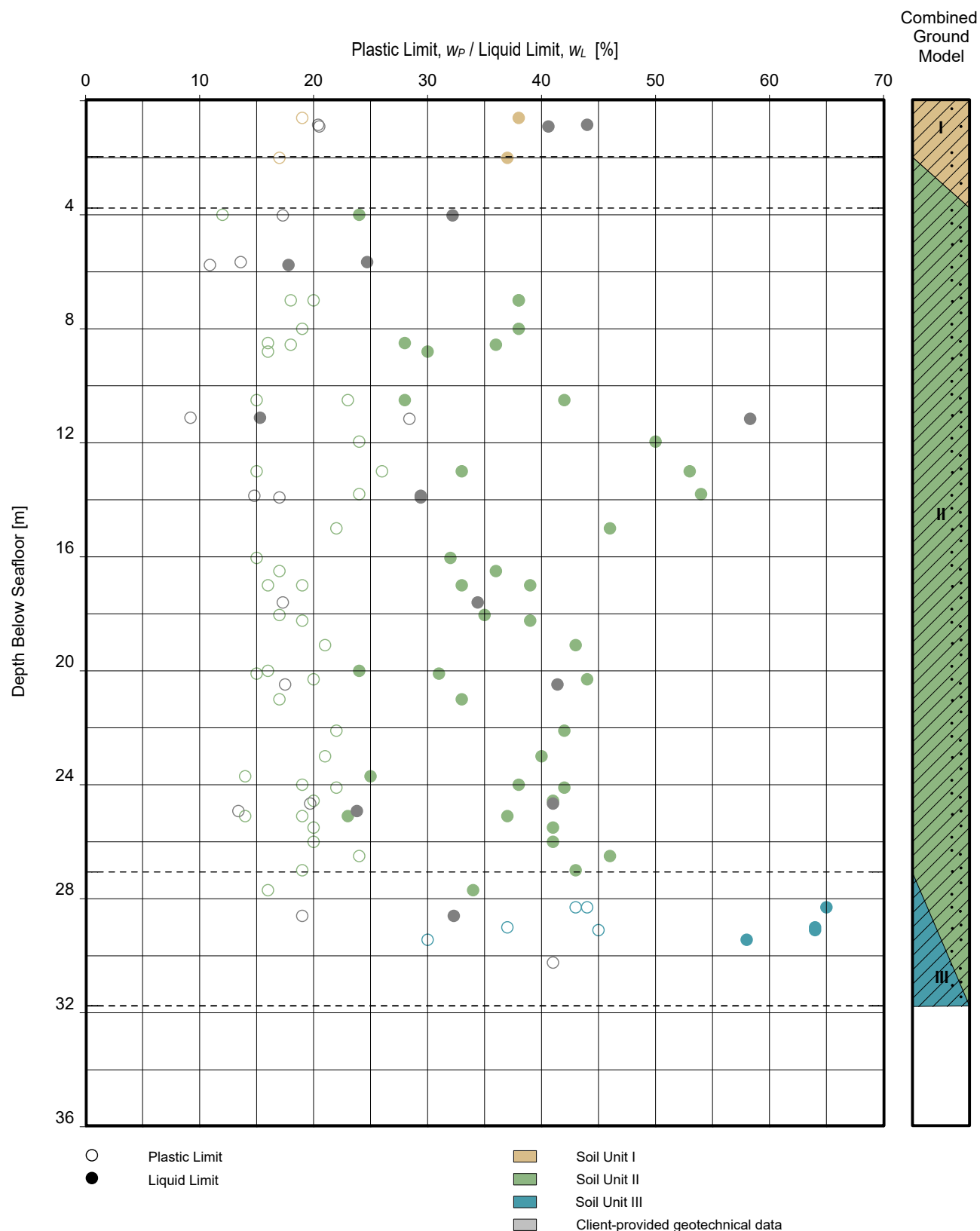
Note(s):
- Refer to Chapter 3 for more information about units and locations represented

WATER CONTENT VERSUS DEPTH



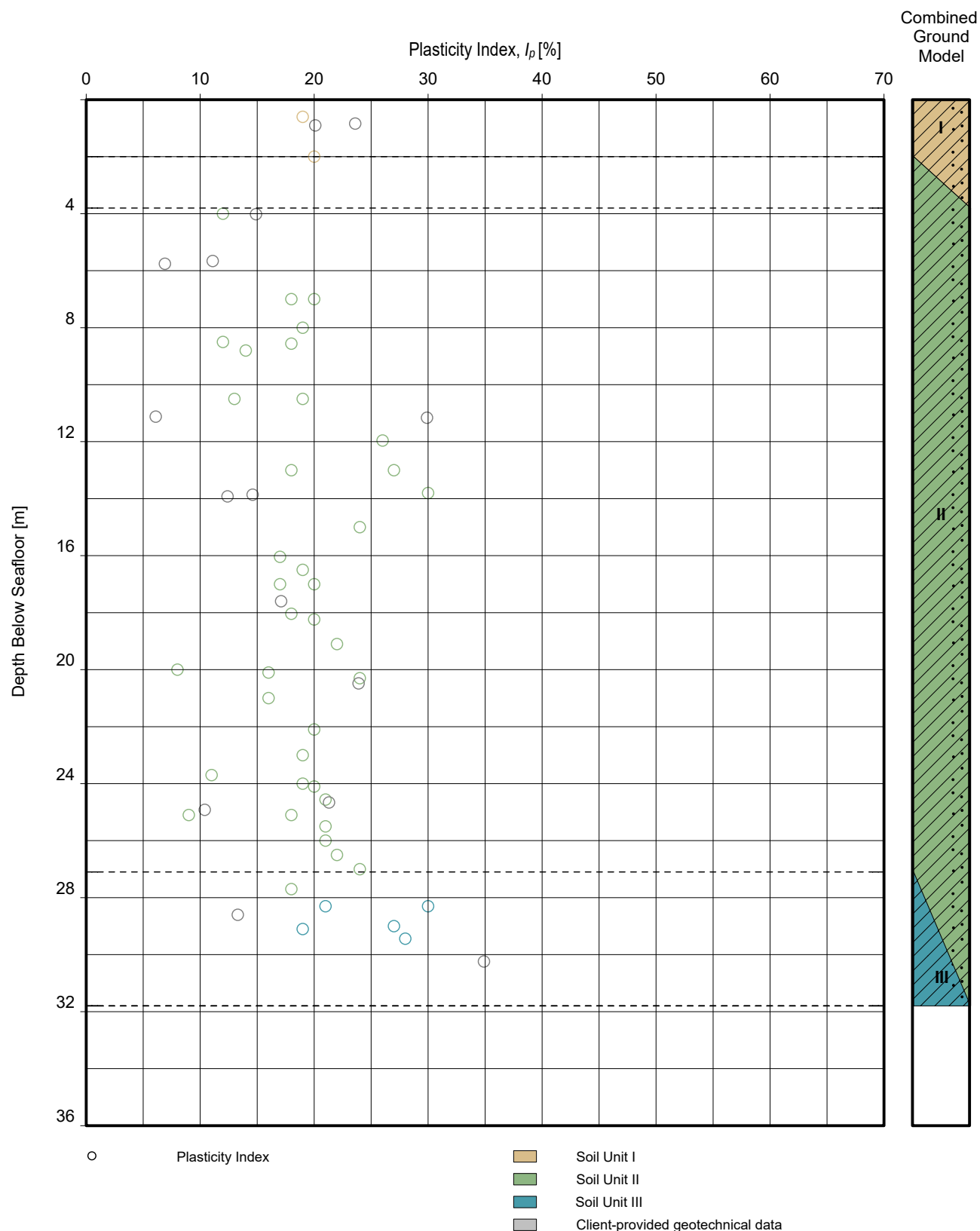
Note(s):
- Refer to Chapter 3 for more information about units and locations represented

WATER CONTENT VERSUS DEPTH



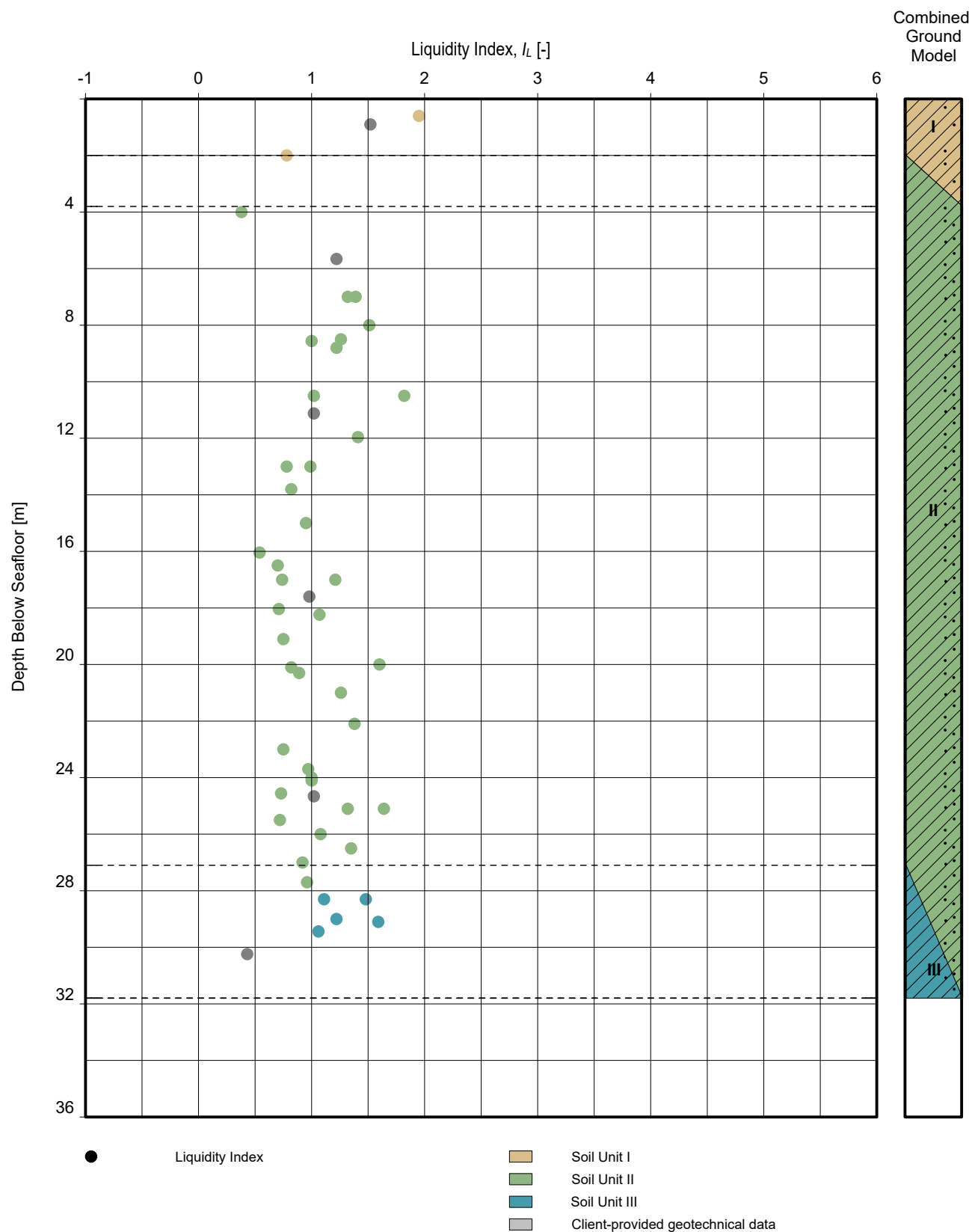
Note(s):
- Refer to Chapter 3 for more information about units and locations represented

PLASTIC LIMIT AND LIQUID LIMIT VERSUS DEPTH



Note(s):
- Refer to Chapter 3 for more information about units and locations represented

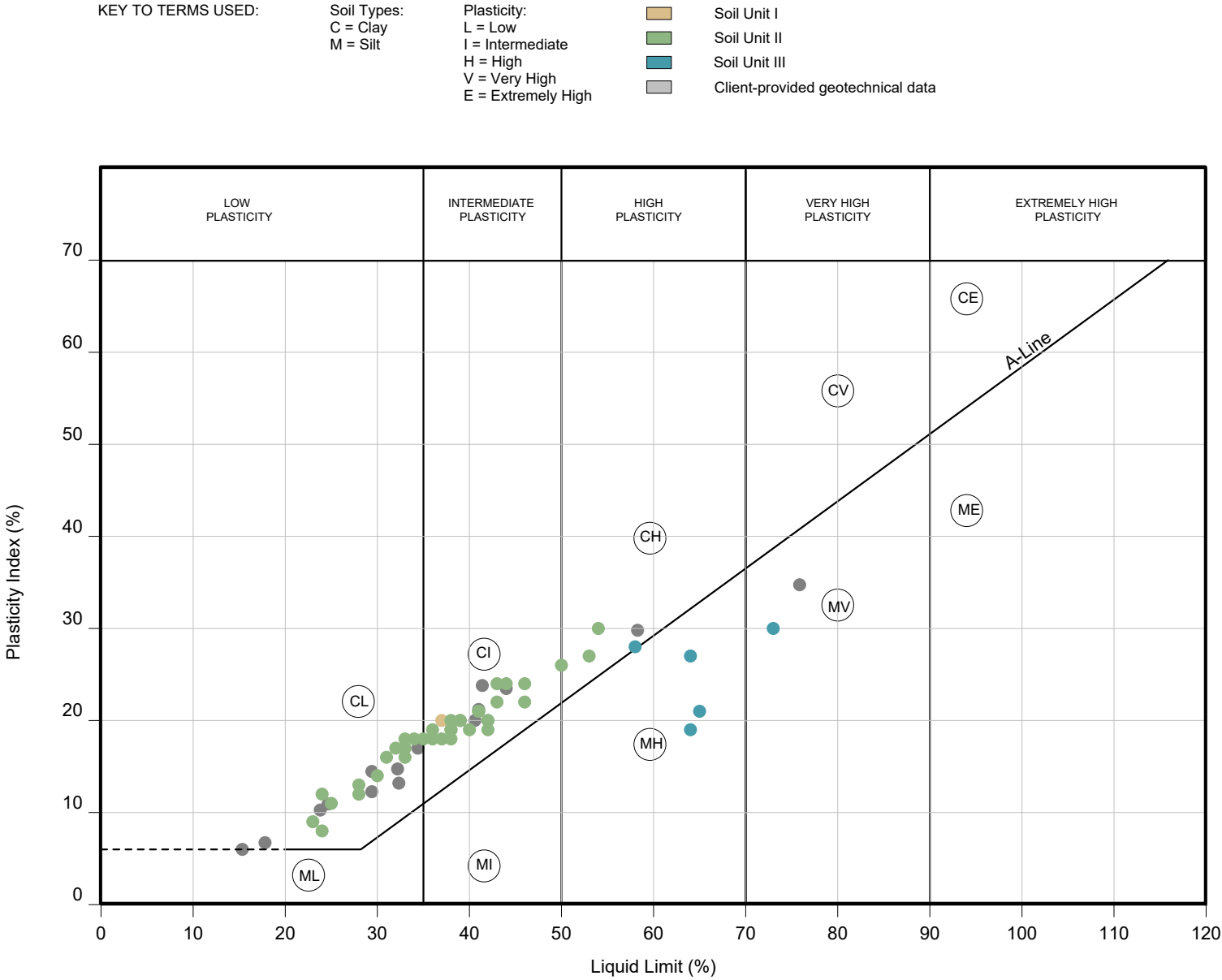
PLASTICITY INDEX VERSUS DEPTH

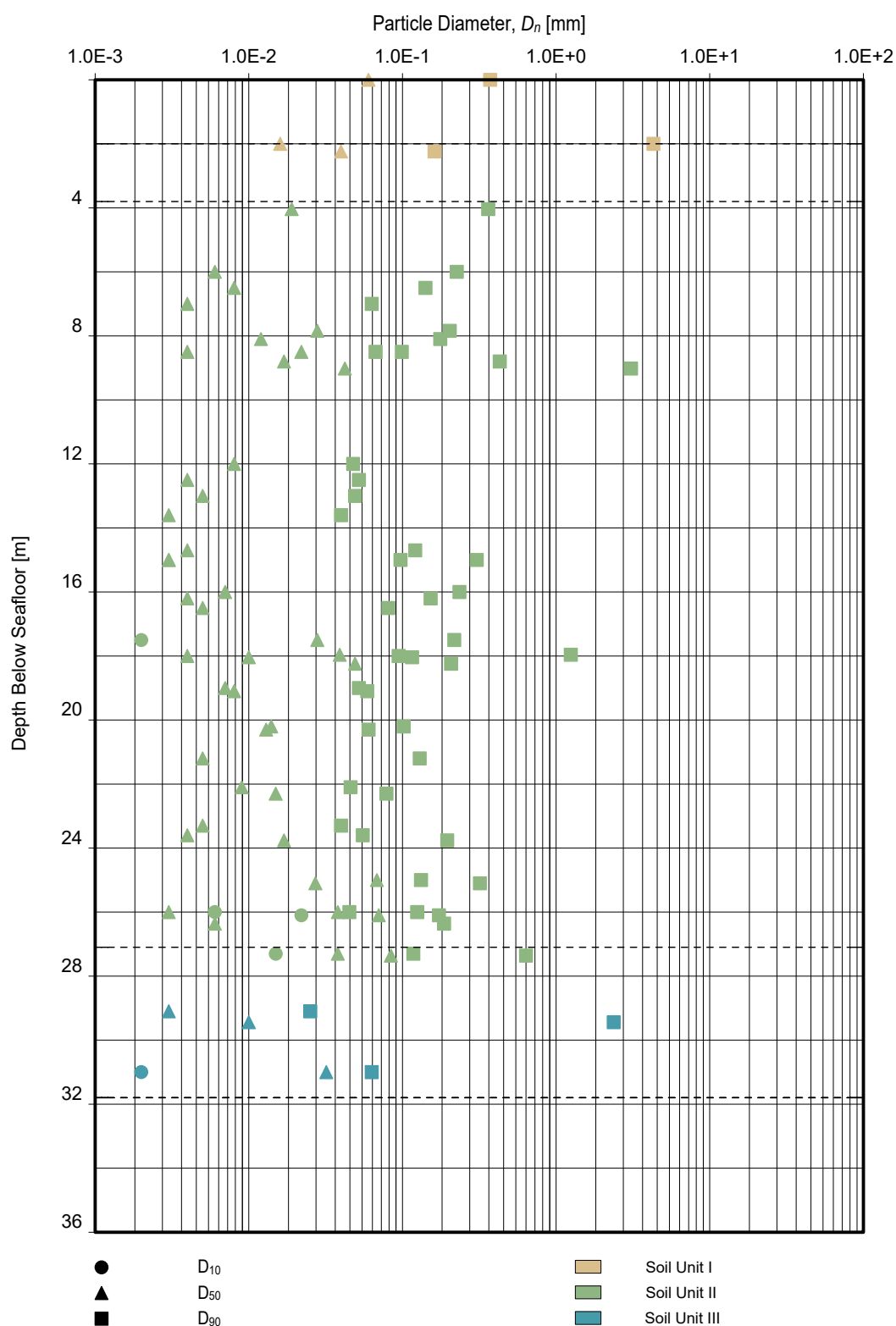


Note(s):
 - Refer to Chapter 3 for more information about units and locations represented

LIQUIDITY INDEX VERSUS DEPTH

PLASTICITY CHART (BS 5930)

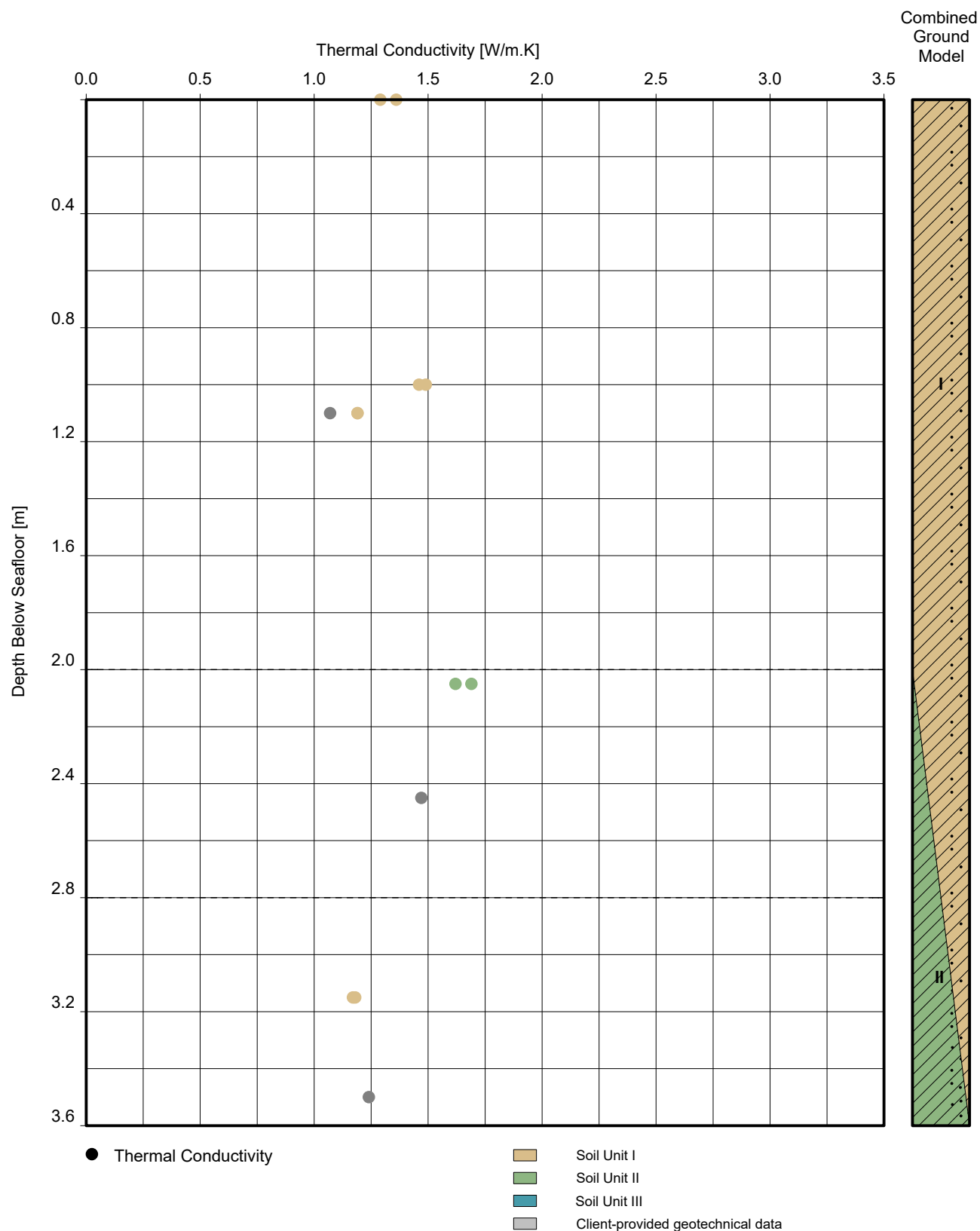




Note(s):

- Refer to Chapter 3 for more information about units and locations represented
- In case D_{10} is not presented, more than 10% of the particles in the tested sample are smaller than 0.002 mm

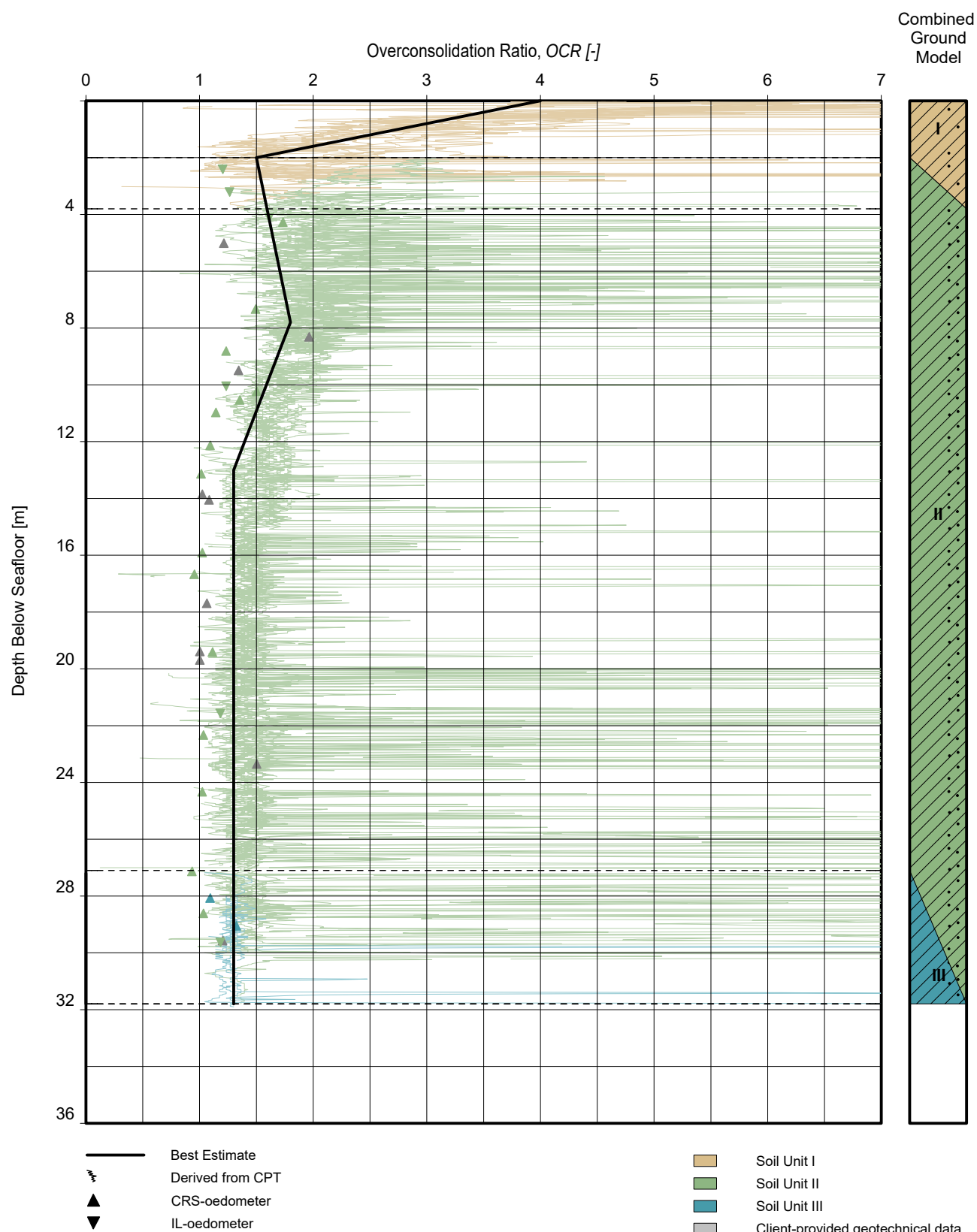
D_{10} D_{50} AND D_{90} VERSUS DEPTH



Note(s):

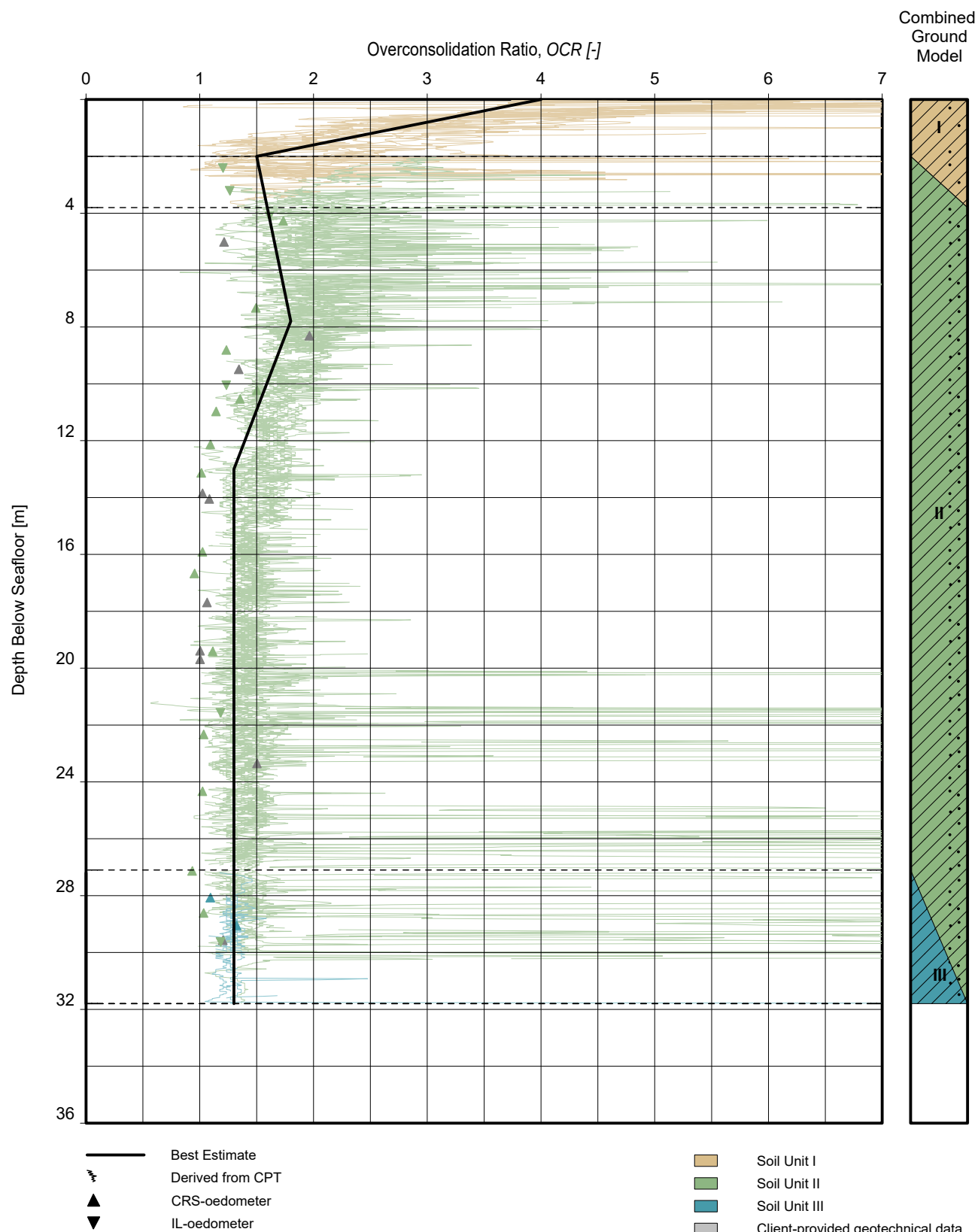
- Refer to Chapter 3 for more information about units and locations represented

THERMAL CONDUCTIVITY VERSUS DEPTH



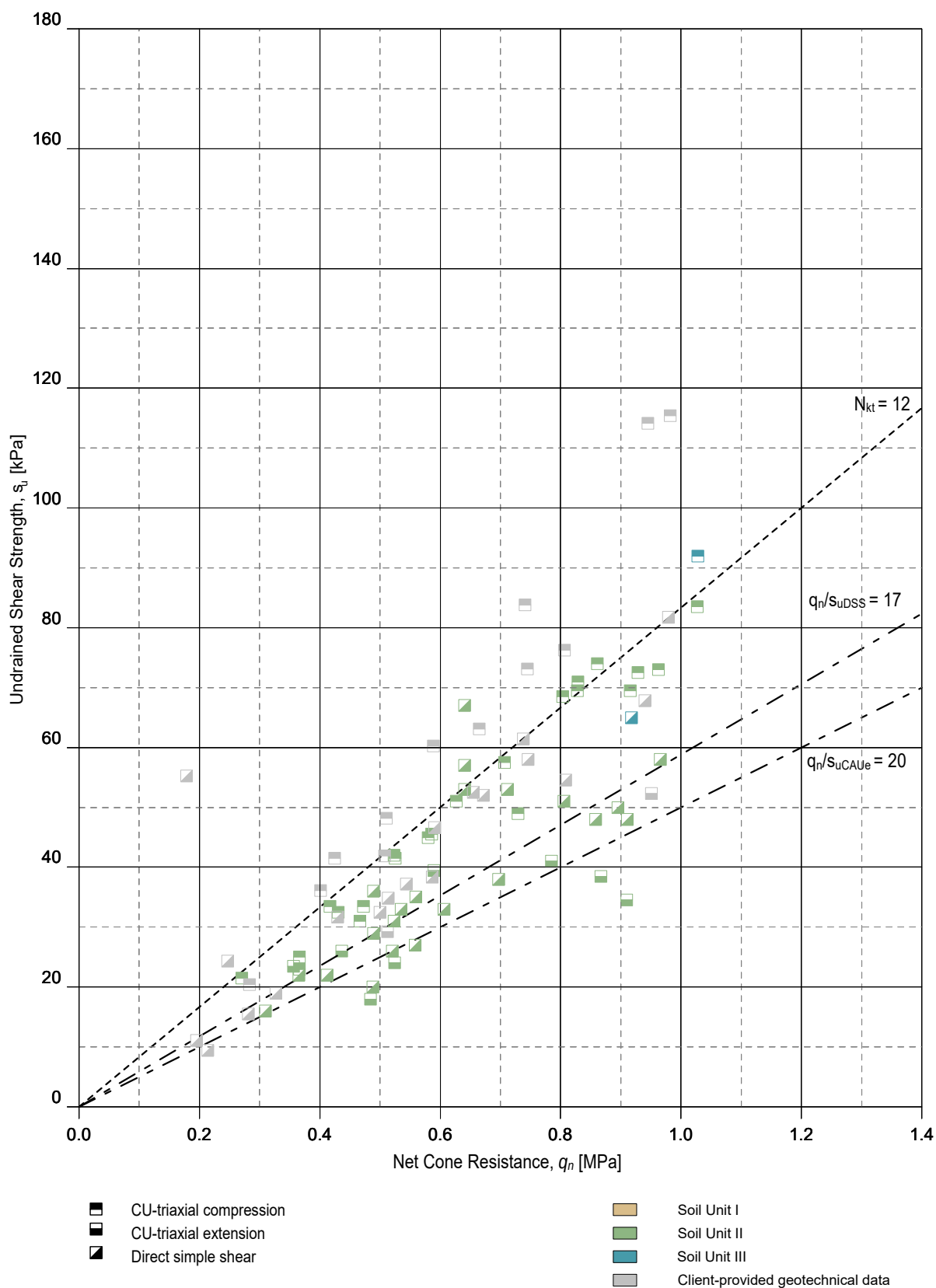
Note(s):
- Refer to Chapter 3 for more information about units and locations represented

OVERCONSOLIDATION RATIO VERSUS DEPTH

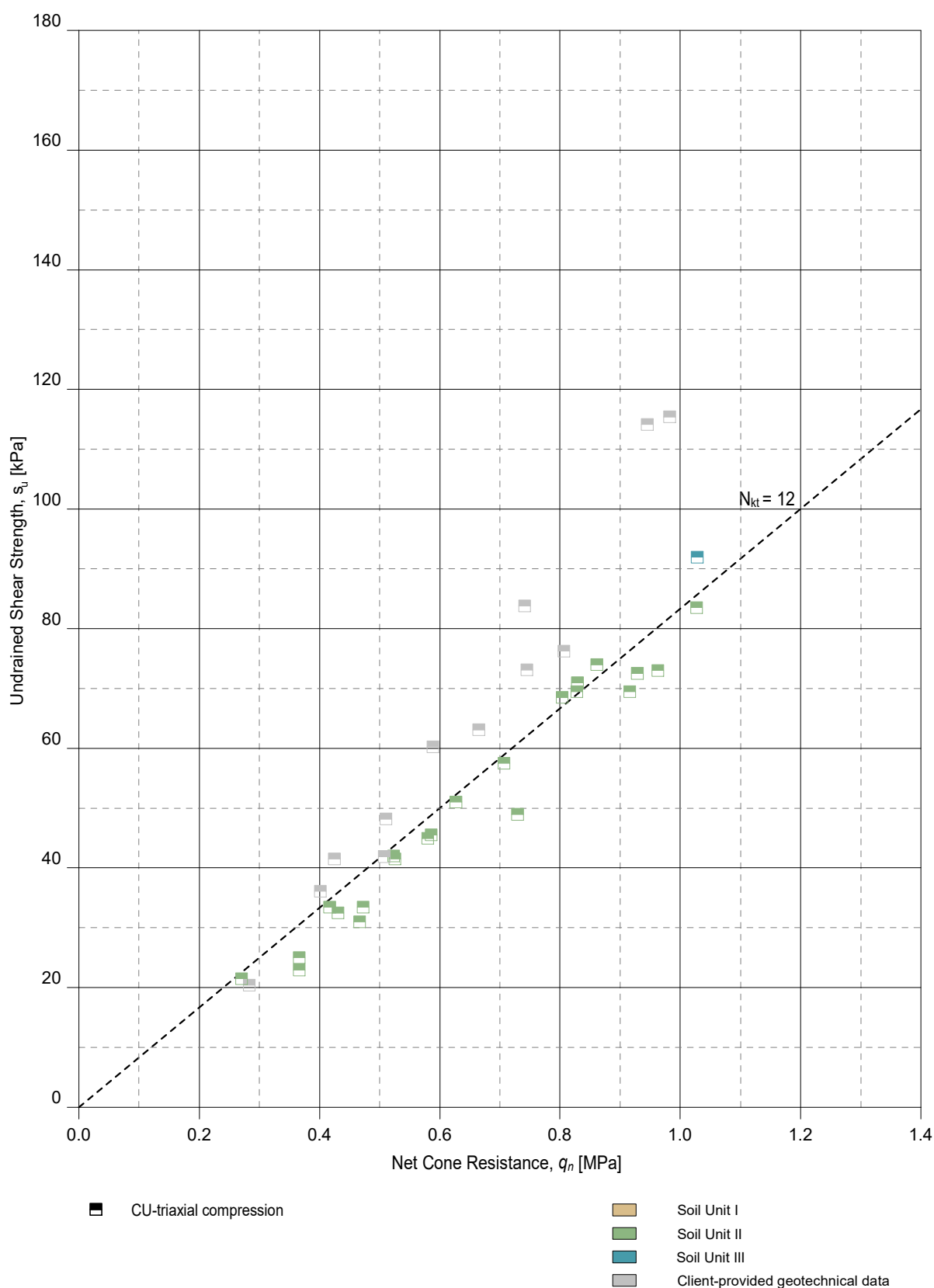


Note(s):
- Refer to Chapter 3 for more information about units and locations represented

OVERCONSOLIDATION RATIO VERSUS DEPTH

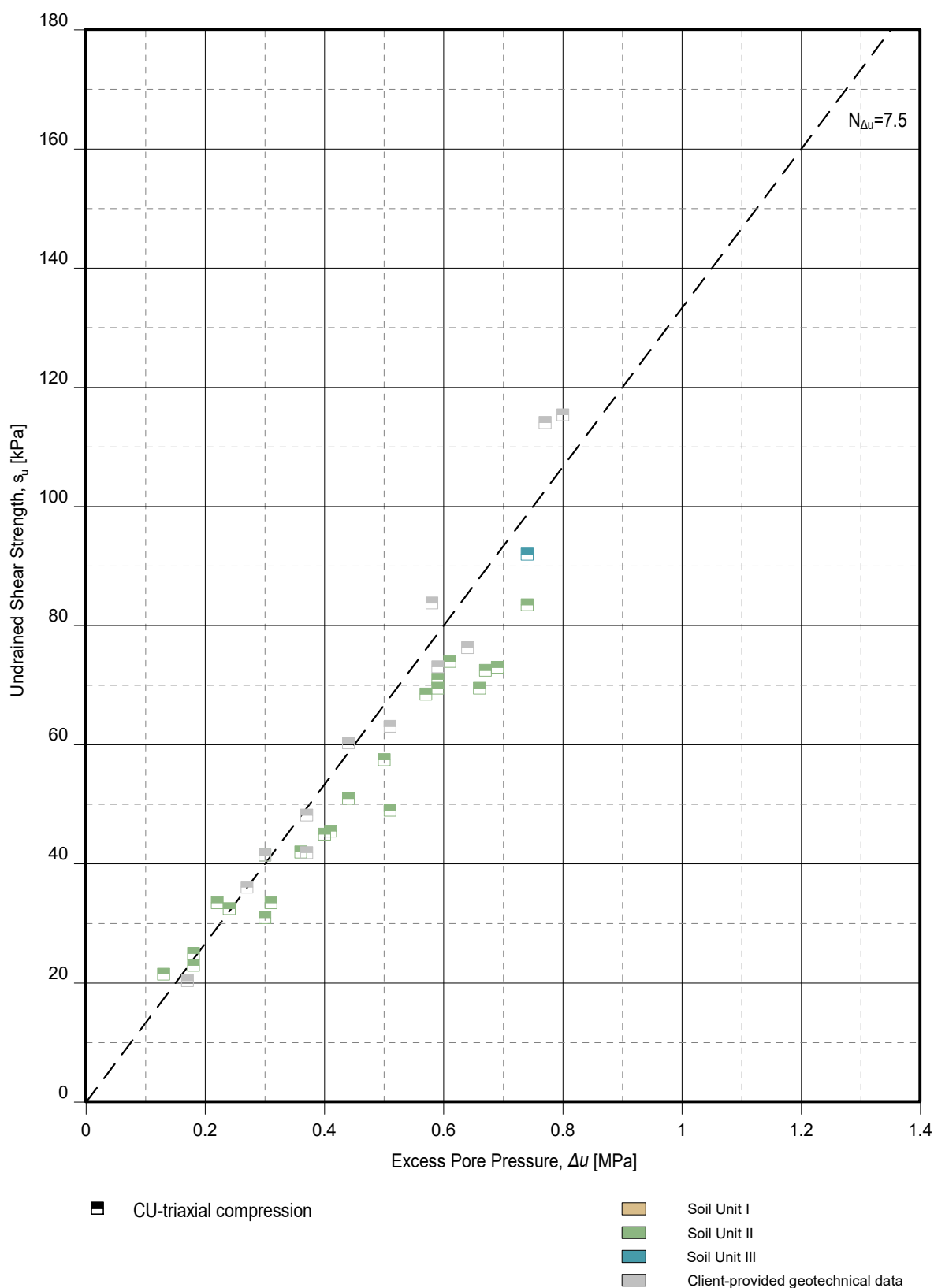


UNDRAINED SHEAR STRENGTH VERSUS NET CONE RESISTANCE

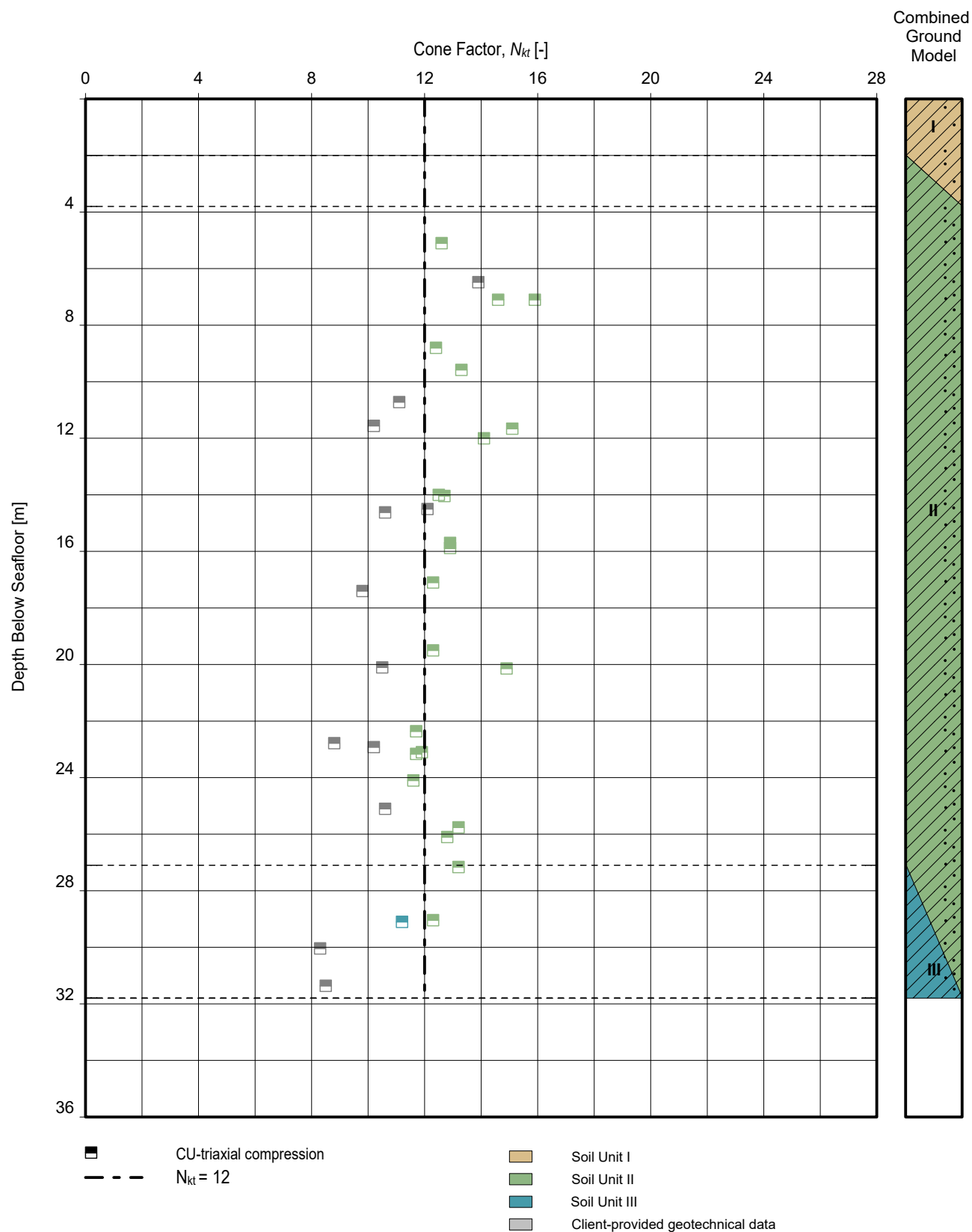


Note(s):
 - Refer to Chapter 3 for more information about units and locations represented
 - CU-triaxial compression test presents s_u derived from q_{max}

UNDRAINED SHEAR STRENGTH VERSUS NET CONE RESISTANCE

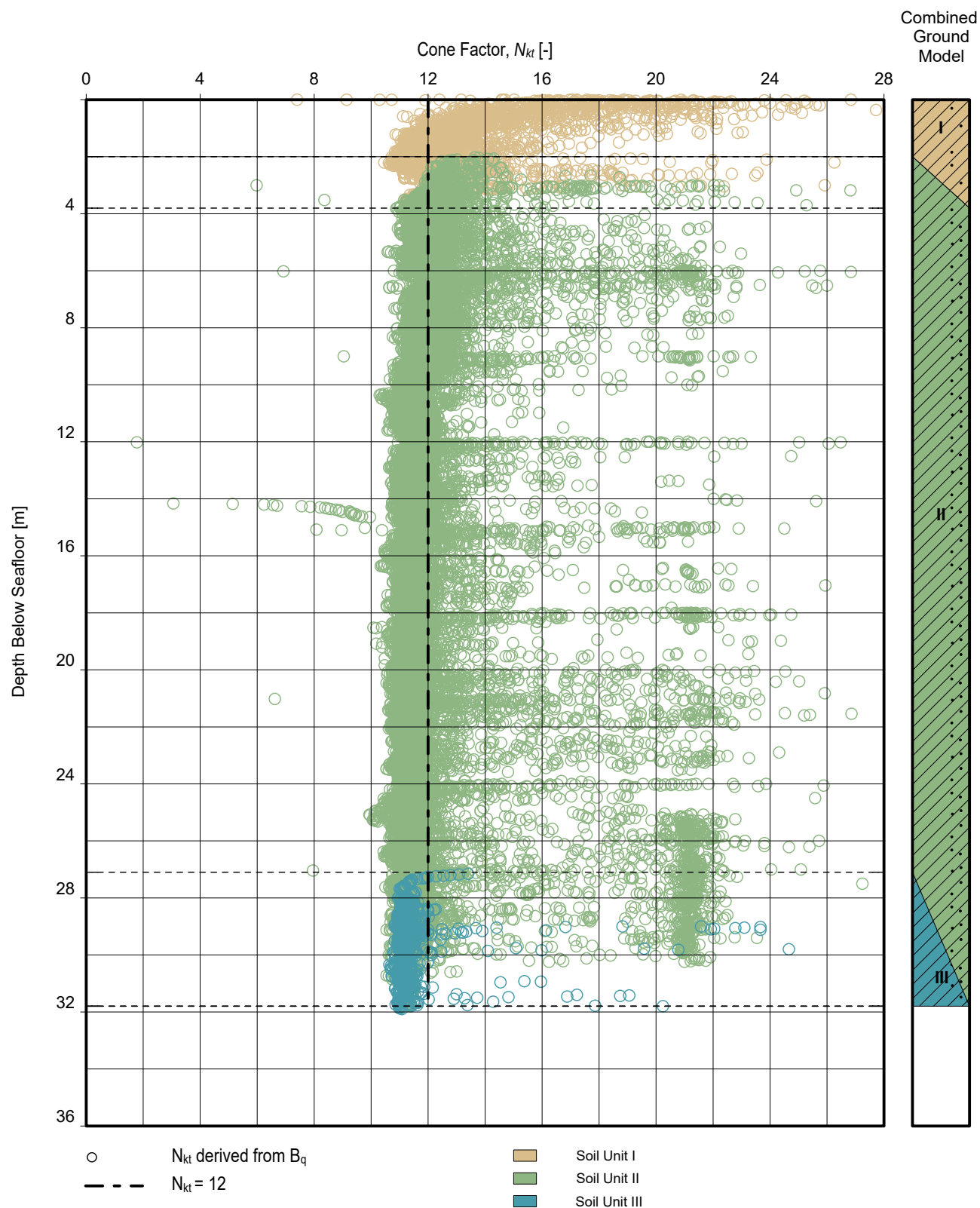


UNDRAINED SHEAR STRENGTH VERSUS EXCESS PORE PRESSURE

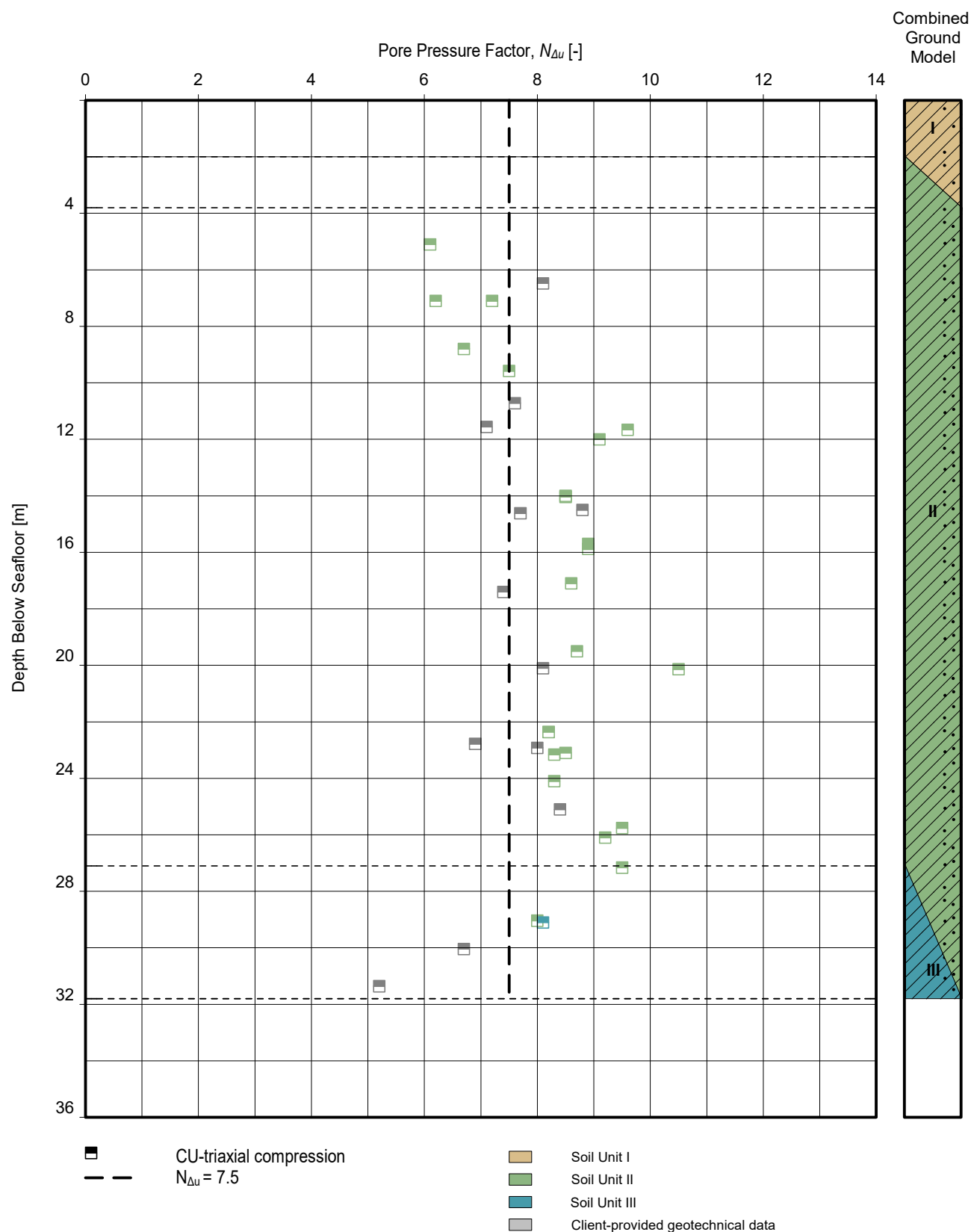


Note(s):
 - Refer to Chapter 3 for more information about units and locations represented

CONE FACTOR VERSUS DEPTH

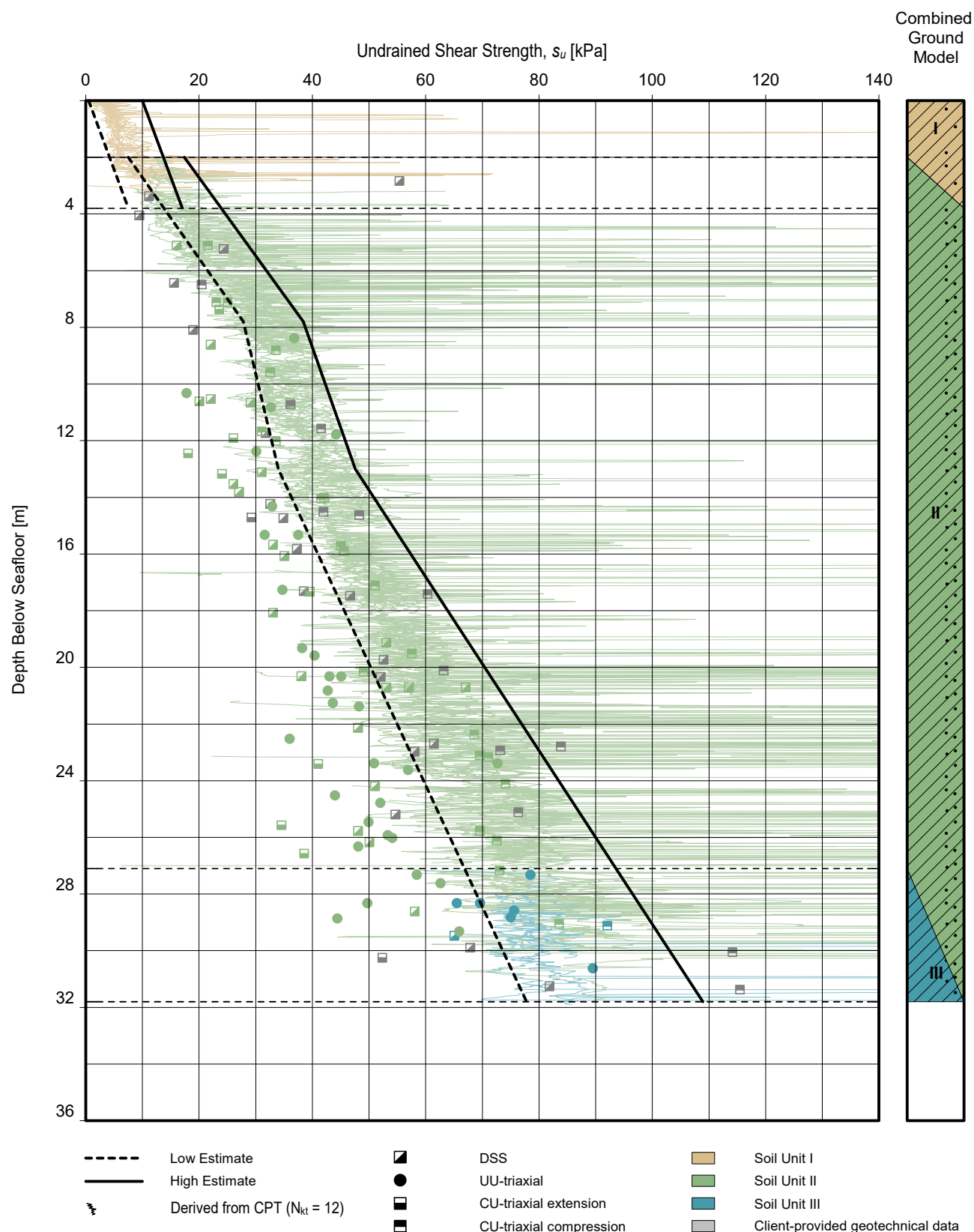


CONE FACTOR DERIVED FROM PORE PRESSURE RATIO VERSUS DEPTH



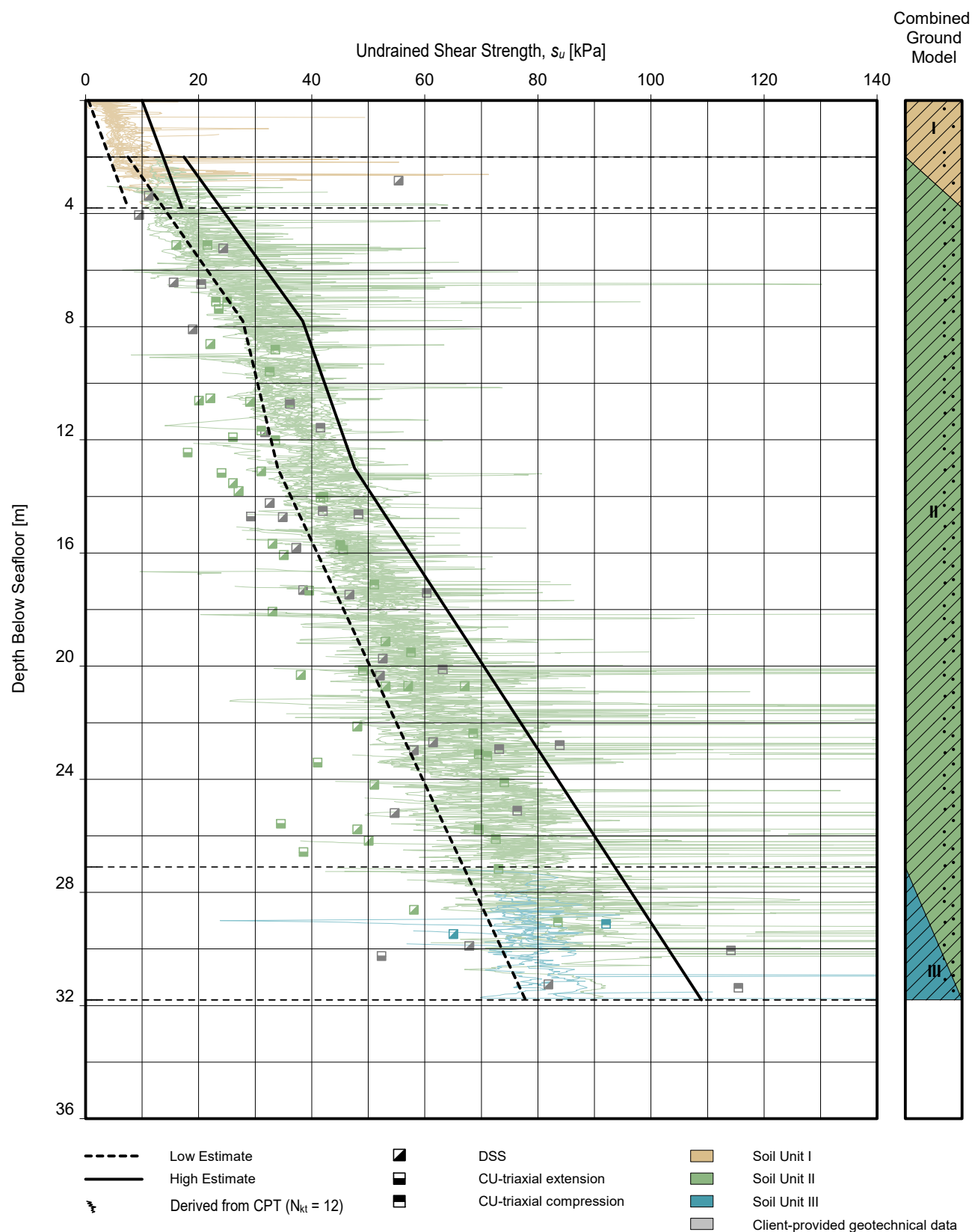
Note(s):
- Refer to Chapter 3 for more information about units and locations represented

PORE PRESSURE FACTOR VERSUS DEPTH



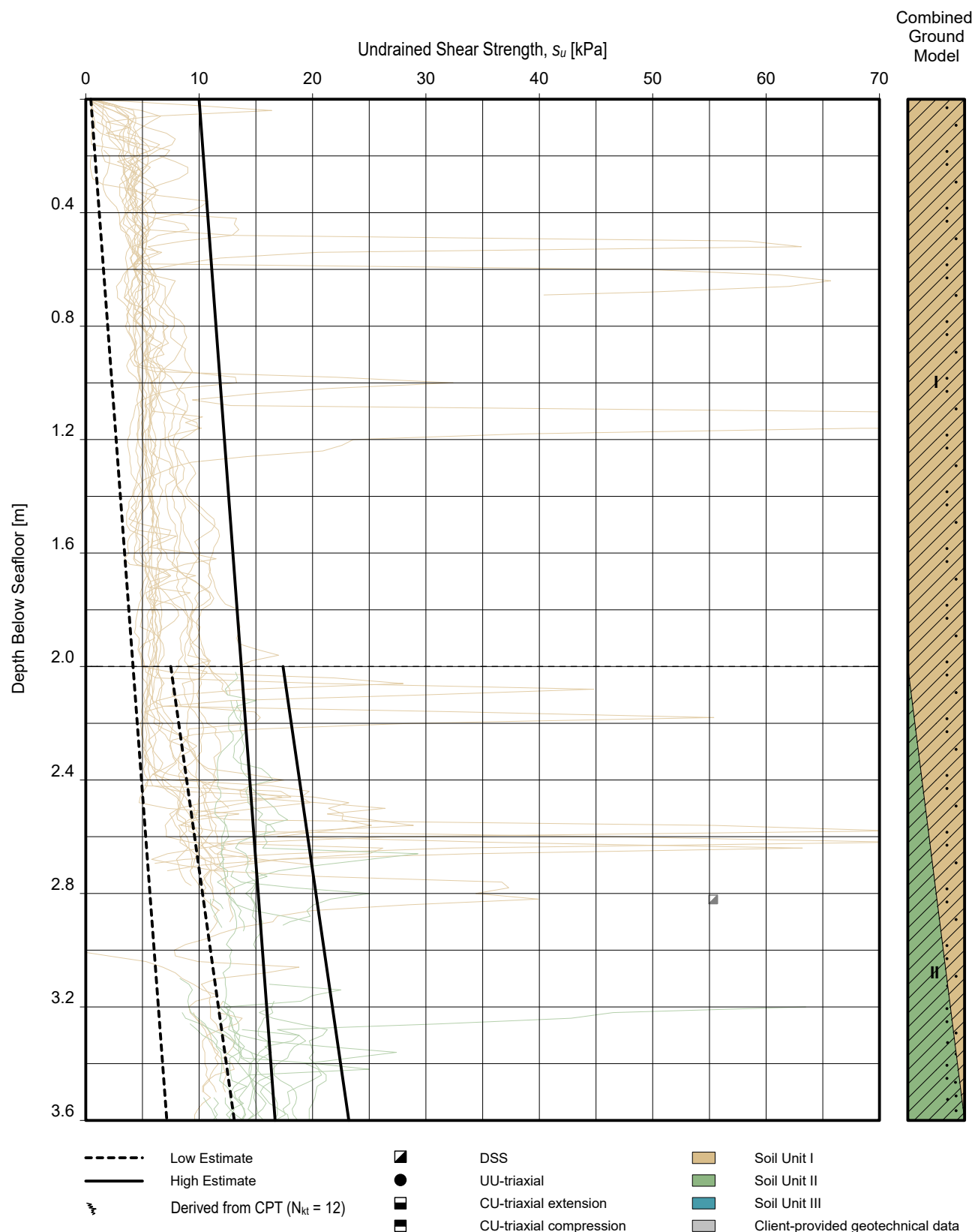
Note(s):
- Refer to Chapter 3 for more information about units and locations represented

UNDRAINED SHEAR STRENGTH VERSUS DEPTH



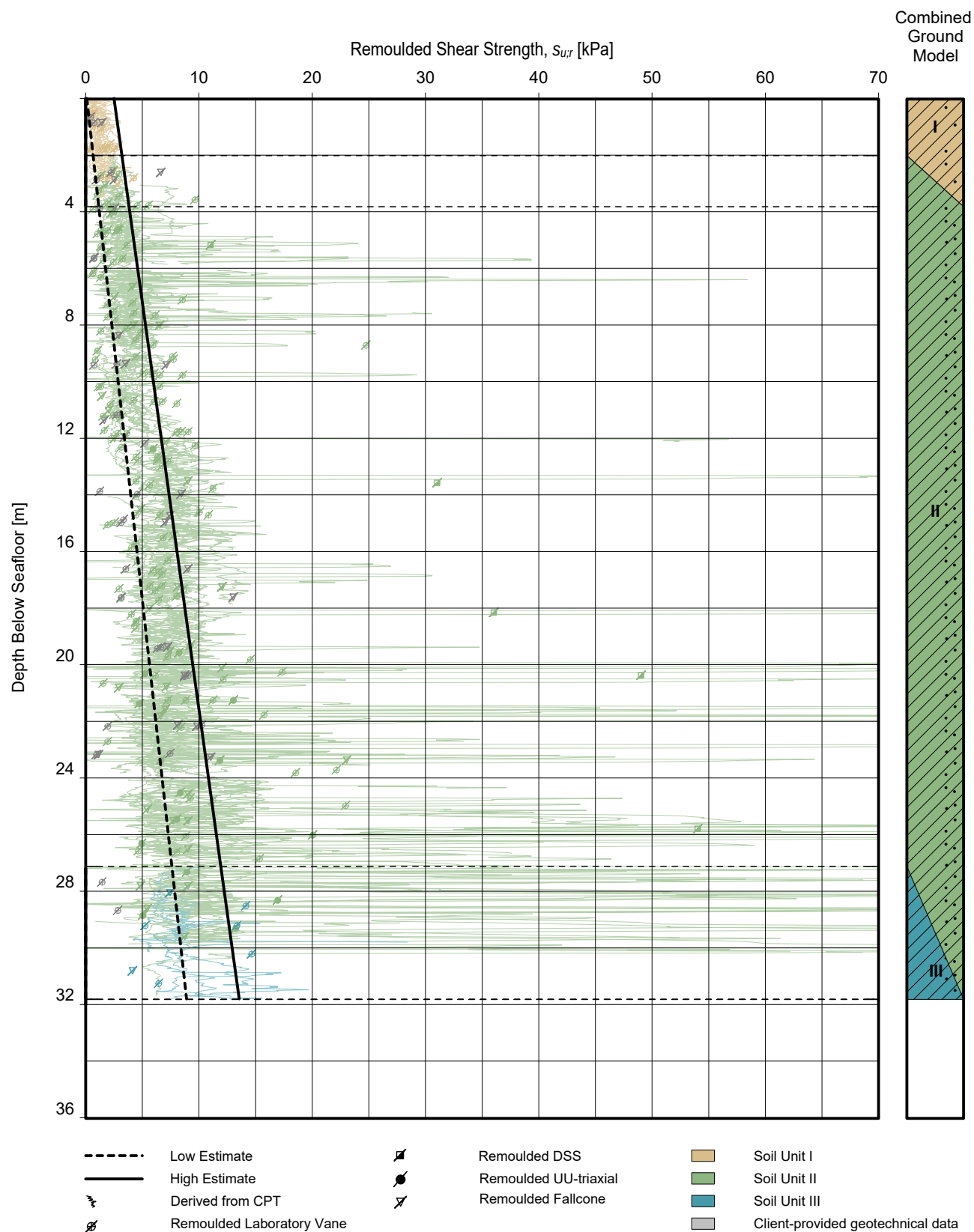
Note(s):
- Refer to Chapter 3 for more information about units and locations represented

UNDRAINED SHEAR STRENGTH VERSUS DEPTH



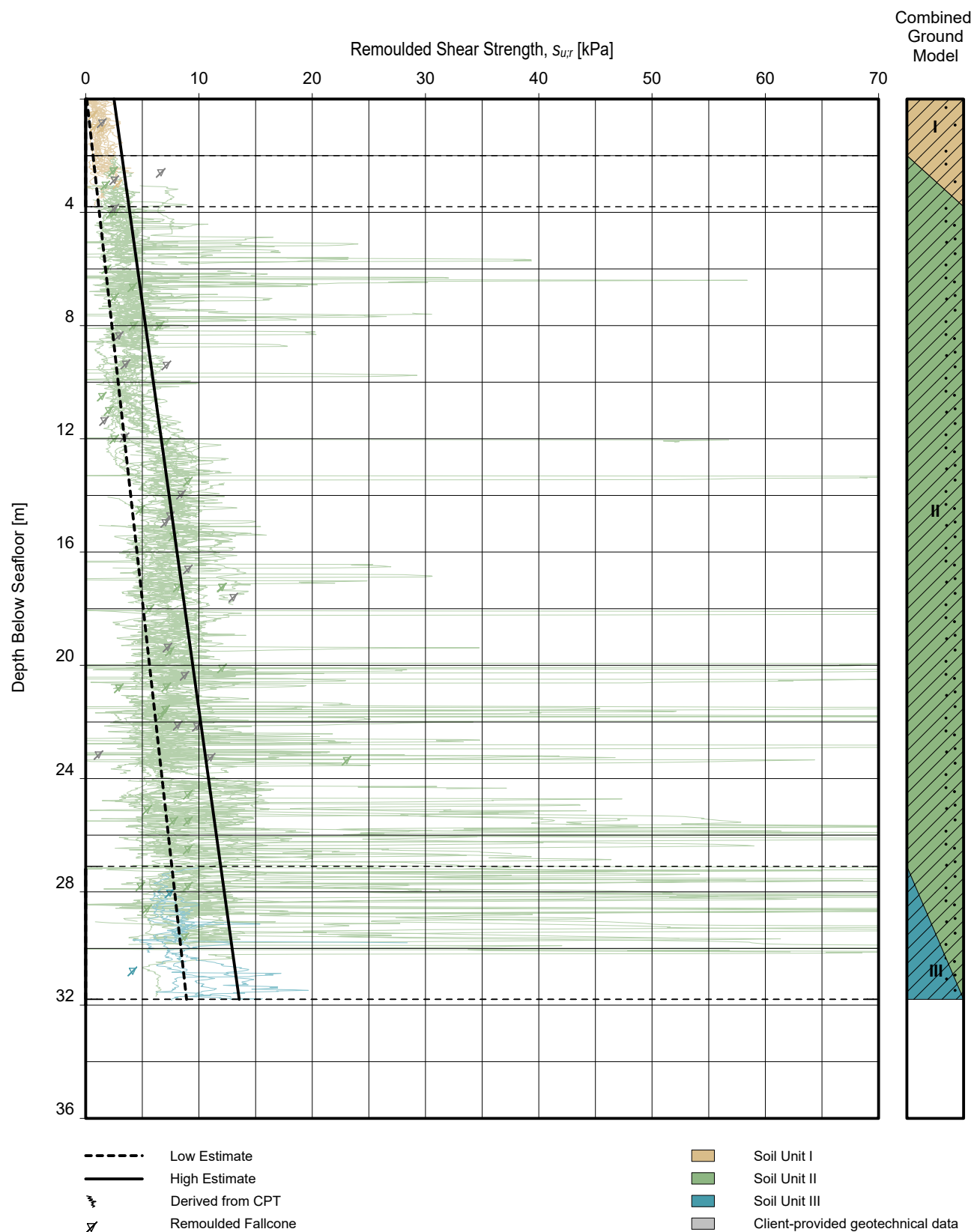
Note(s):
- Refer to Chapter 3 for more information about units and locations represented

UNDRAINED SHEAR STRENGTH VERSUS DEPTH

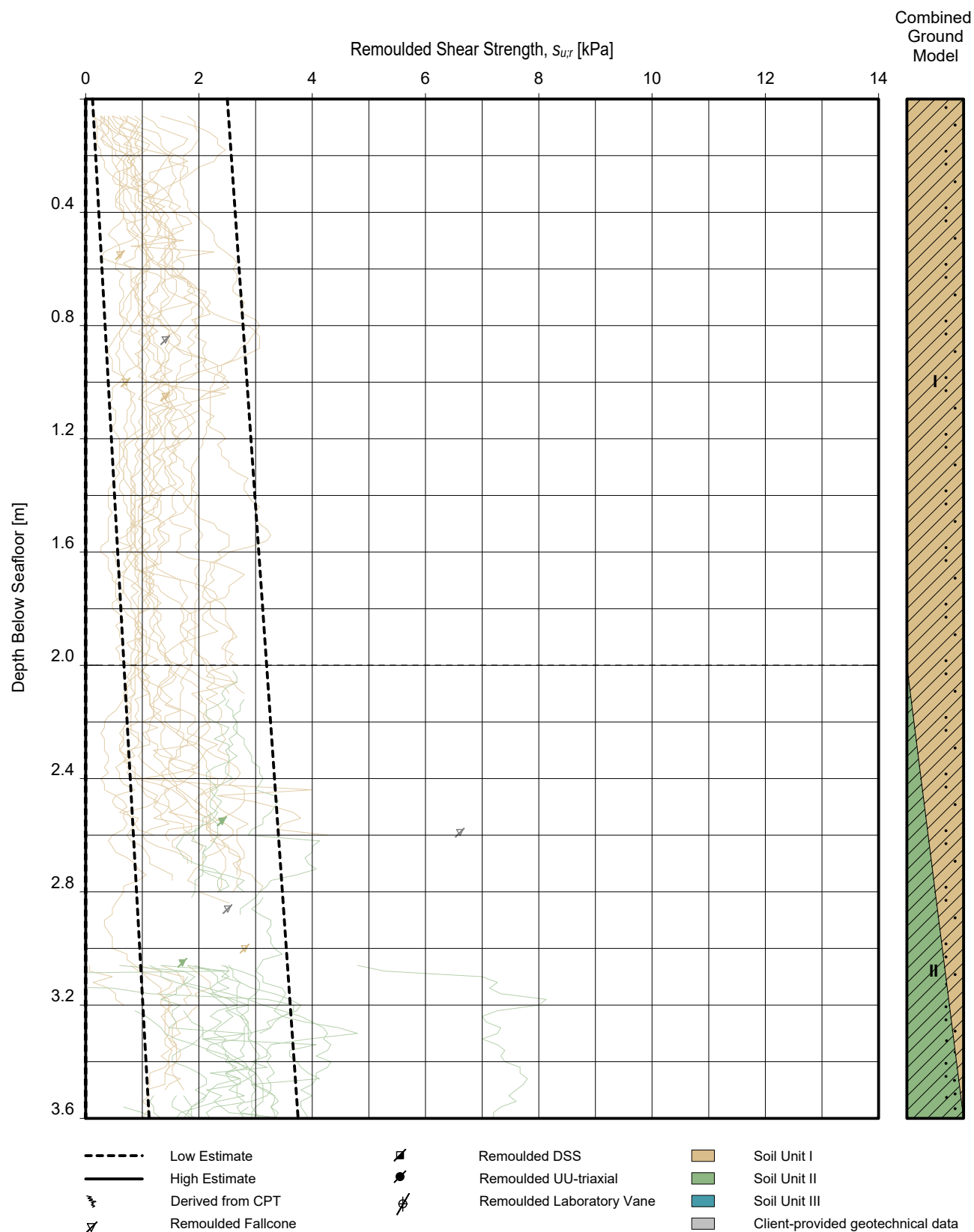


Note(s):
- Refer to Chapter 3 for more information about units and locations represented

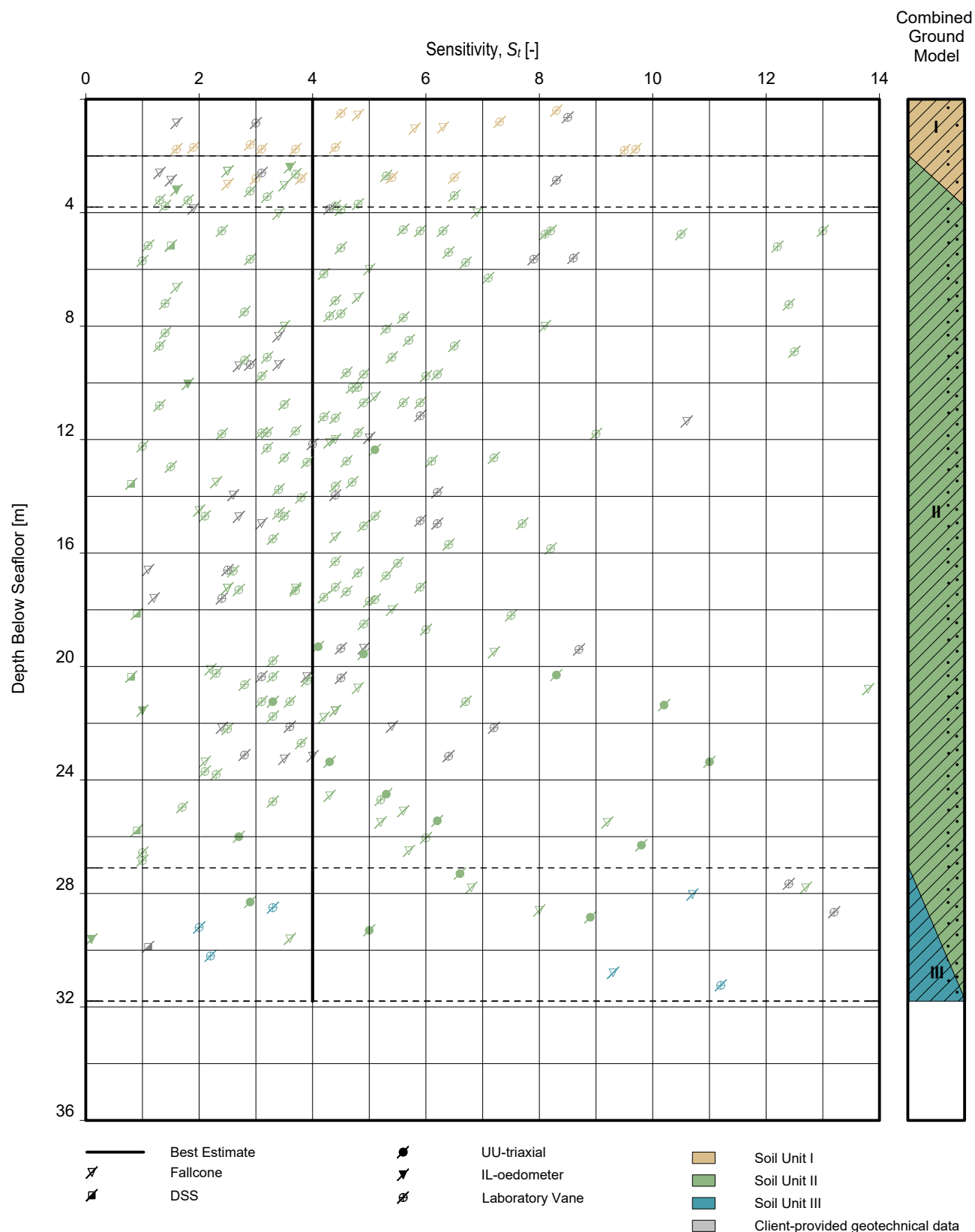
REMOULDED SHEAR STRENGTH VERSUS DEPTH



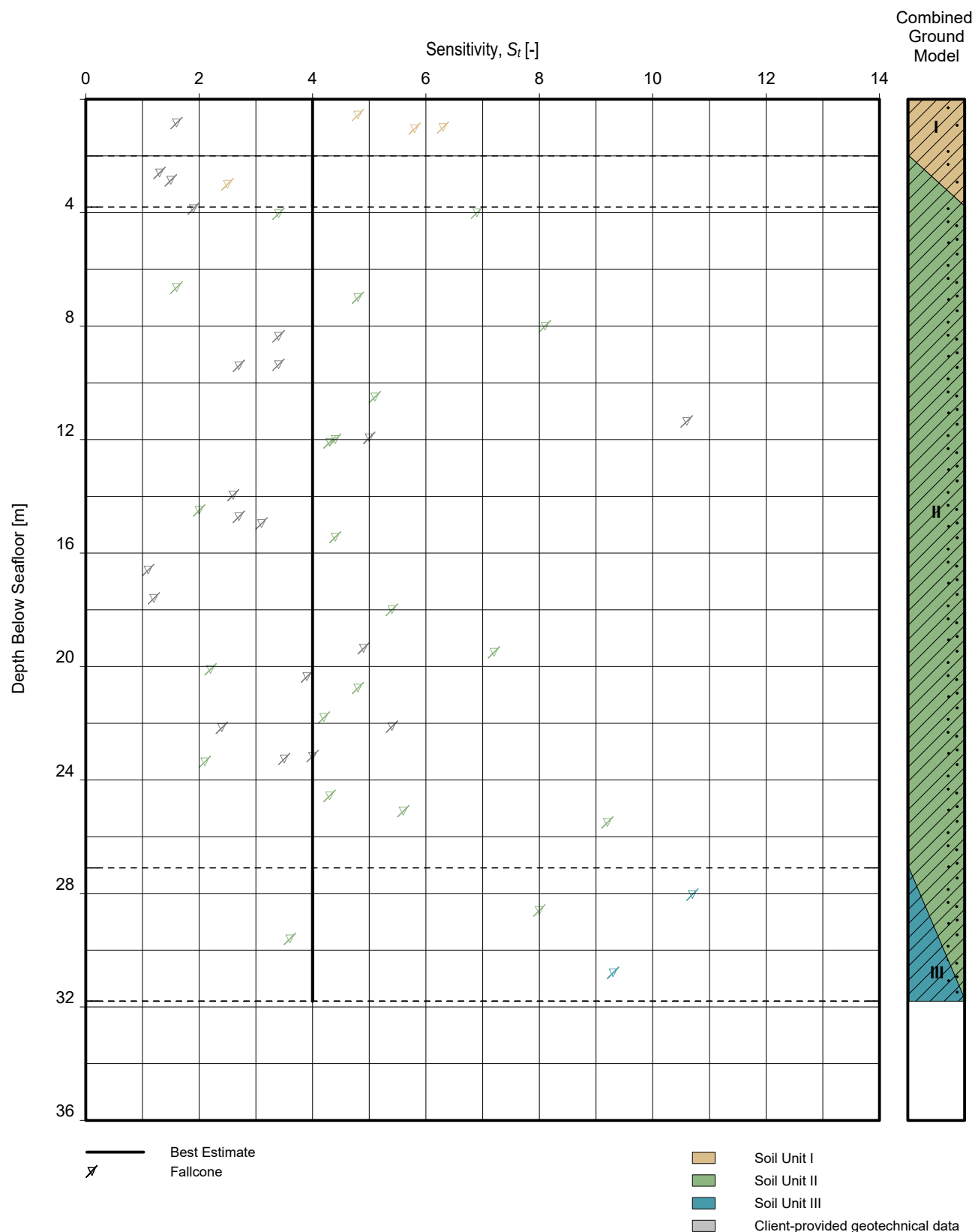
REMOULDED SHEAR STRENGTH VERSUS DEPTH



REMOULDED SHEAR STRENGTH VERSUS DEPTH



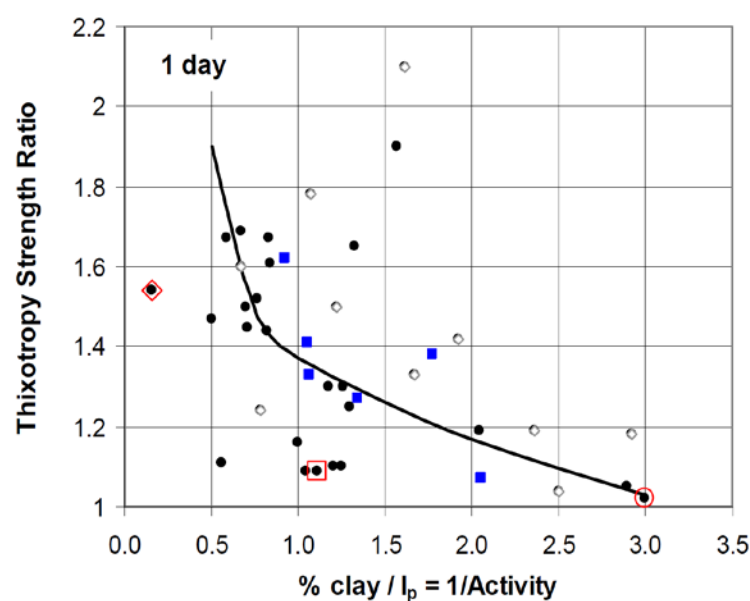
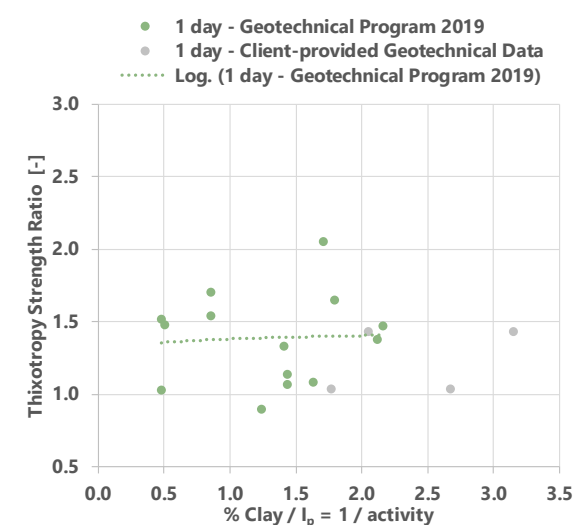
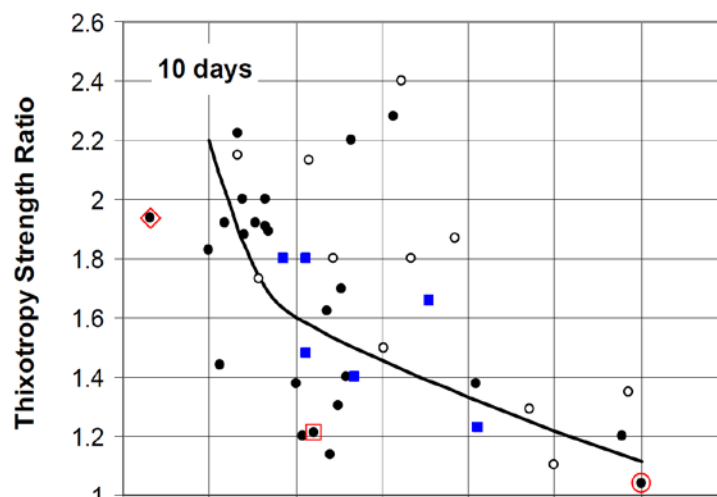
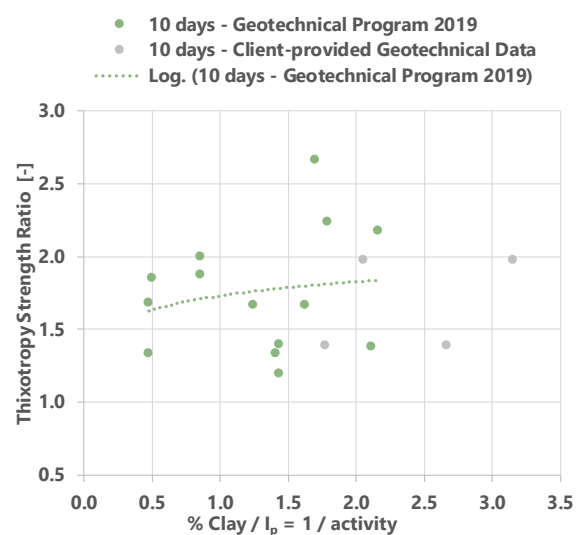
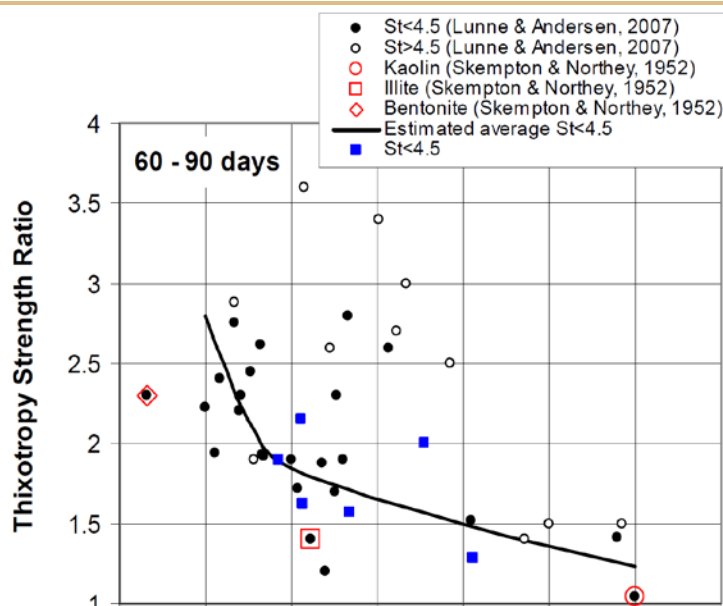
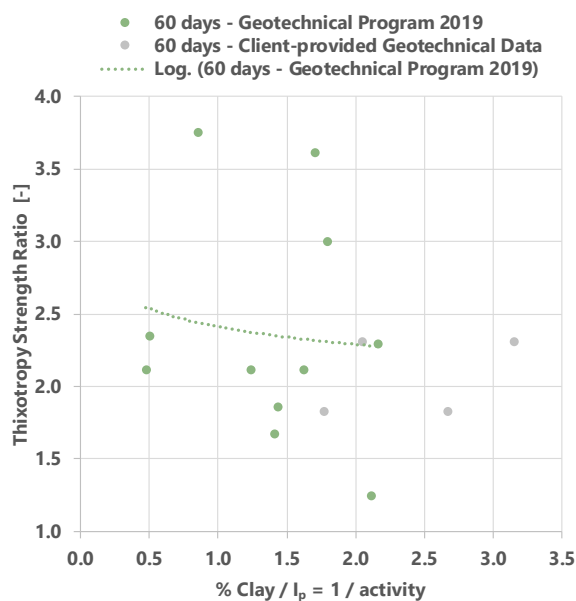
SENSITIVITY VERSUS DEPTH



Note(s):
 - Refer to Chapter 3 for more information about units and locations represented

SENSITIVITY VERSUS DEPTH

BAY DU NORD GP 2019 - ATLANTIC OCEAN, CANADA

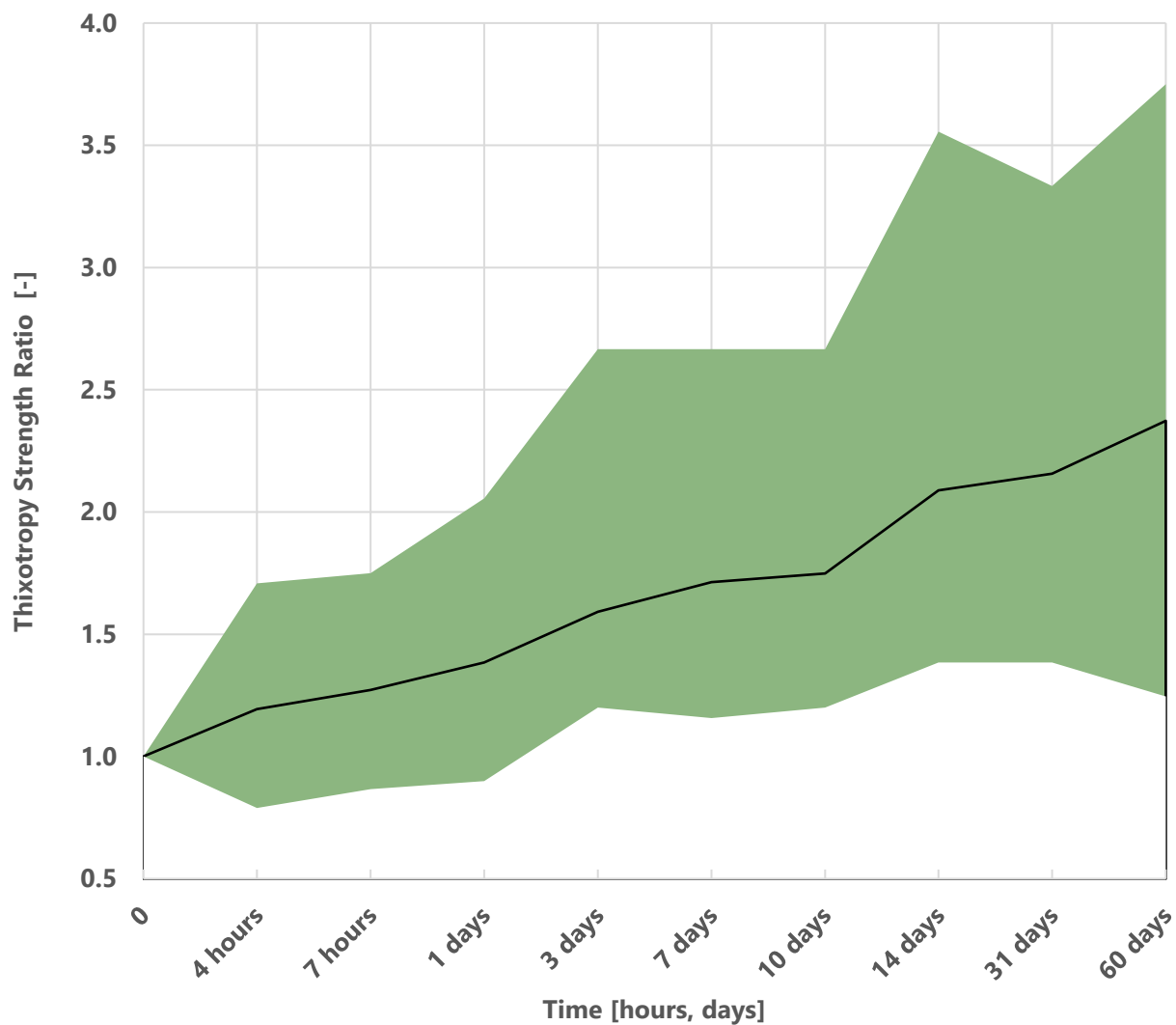


Notes:

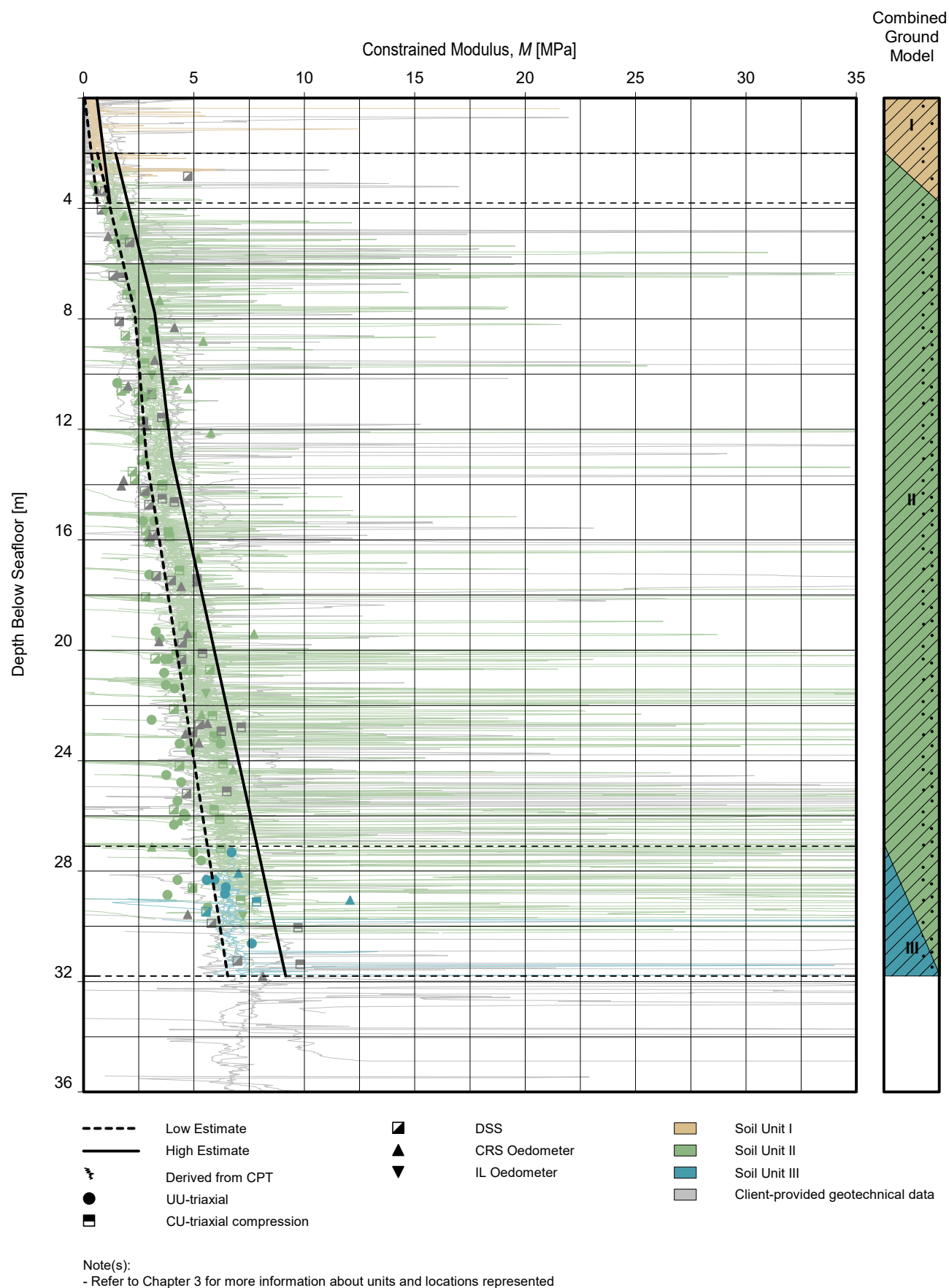
- Right-hand plots obtained from Andersen (2008).

THIXOTROPY STRENGTH RATIO VERSUS INVERSE ACTIVITY

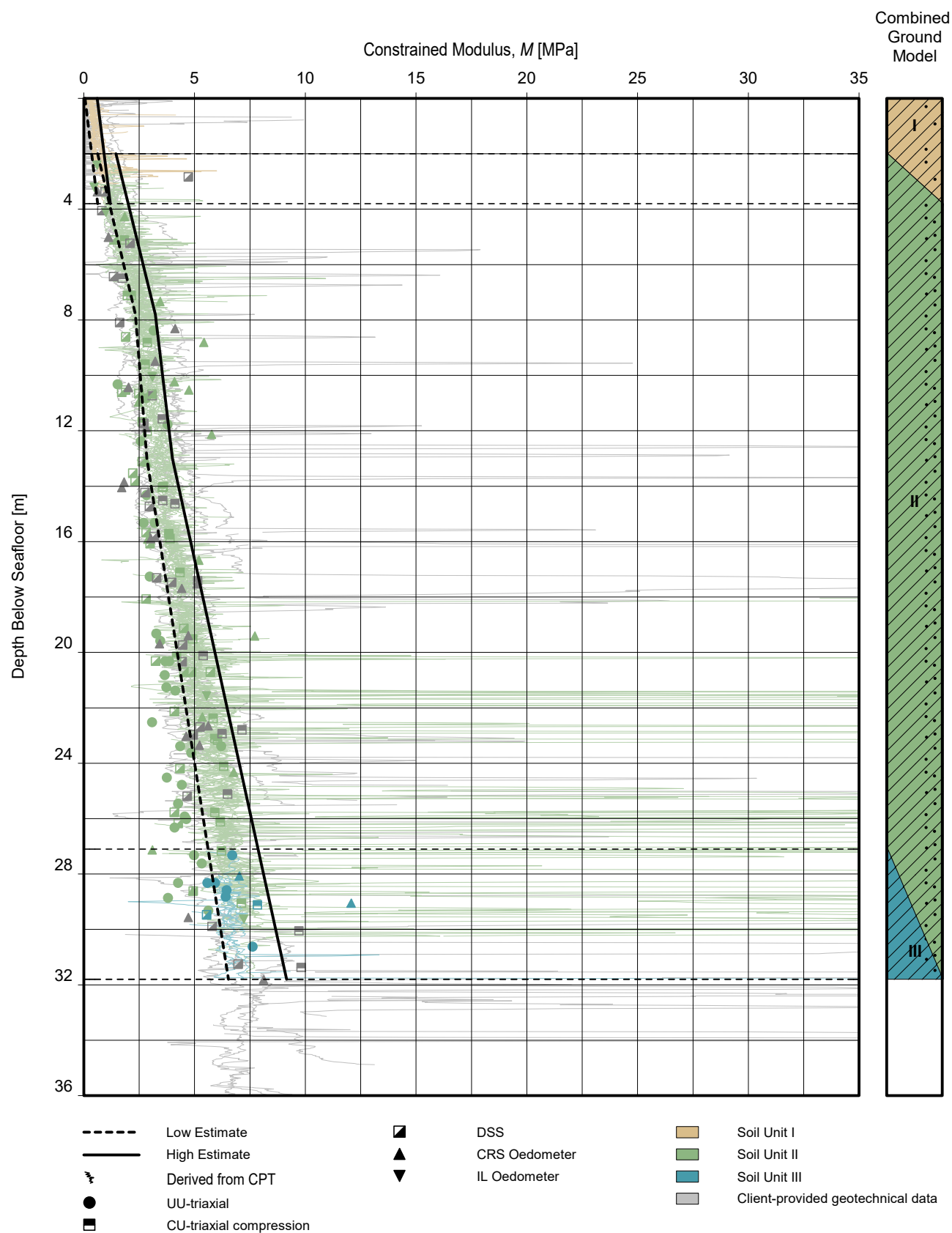
Minimum, maximum and mean thixotropy strength ratio observed over time



THIXOTROPY STRENGTH RATIO VERSUS TIME



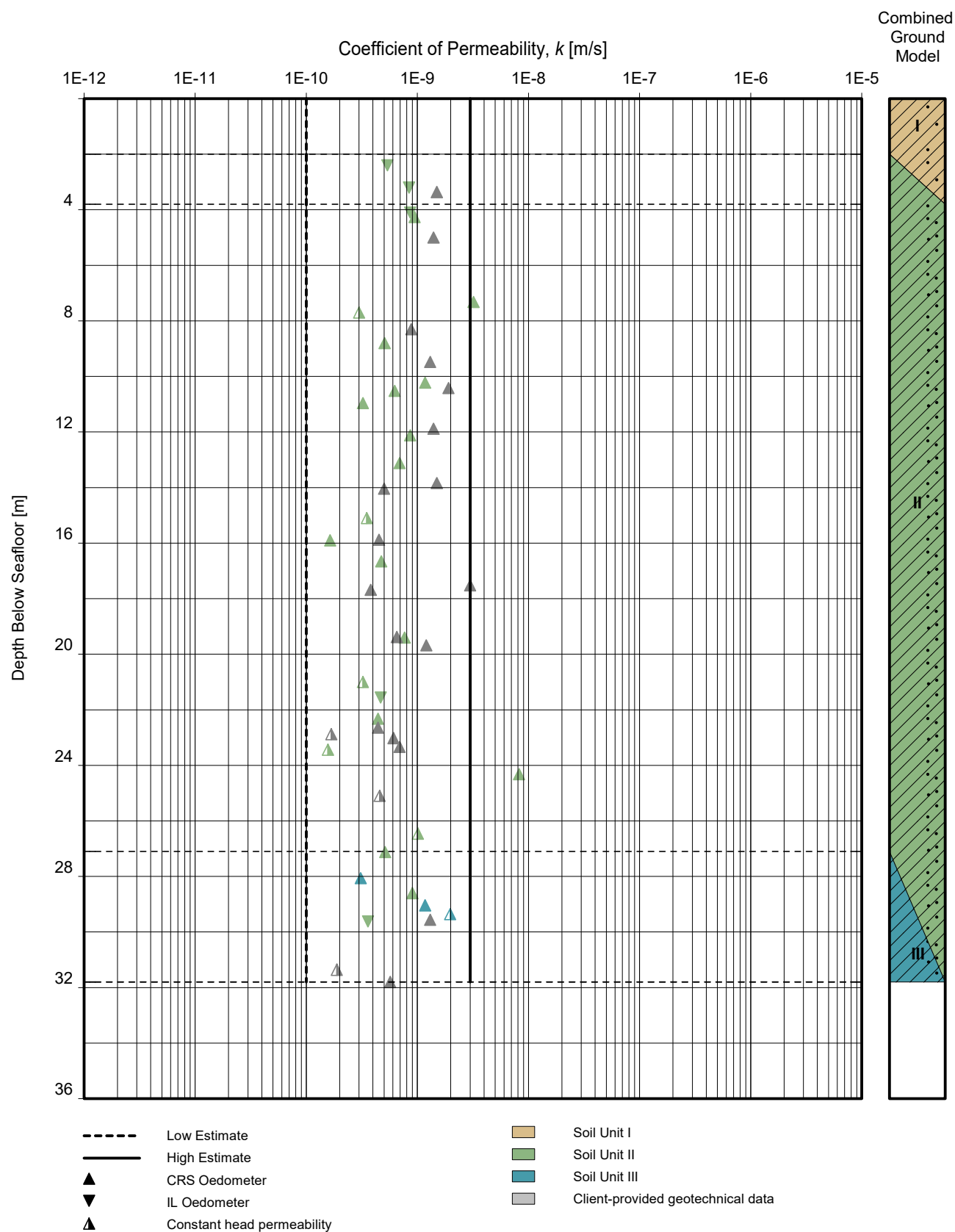
CONSTRAINED MODULUS VERSUS DEPTH



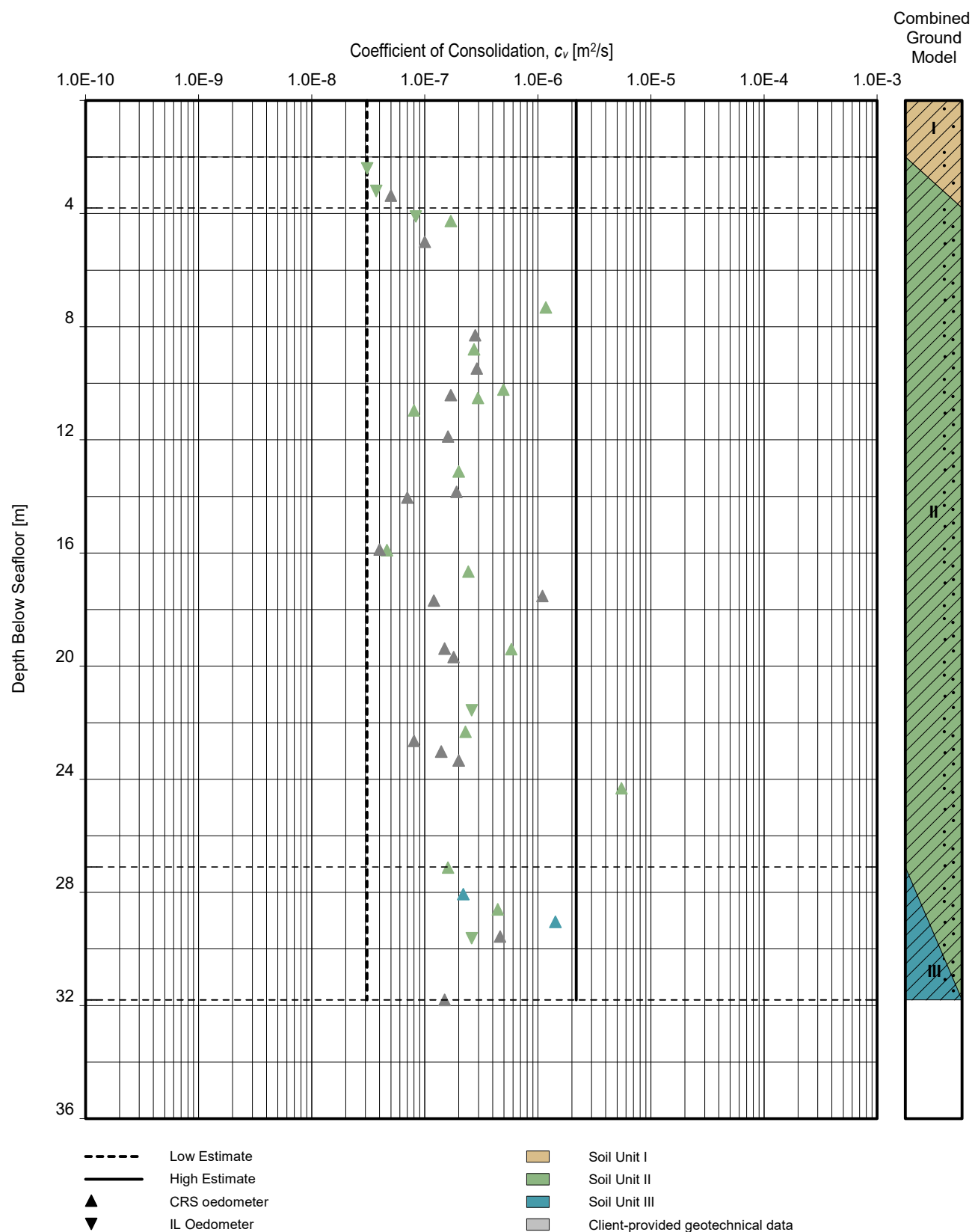
Note(s):
- Refer to Chapter 3 for more information about units and locations represented

CONSTRAINED MODULUS VERSUS DEPTH

GeODir/3.8-1a_Constrained Modulus vs Depth_Cleaned Version_85 multiplier.GGF/2020-04-17 07:48:19

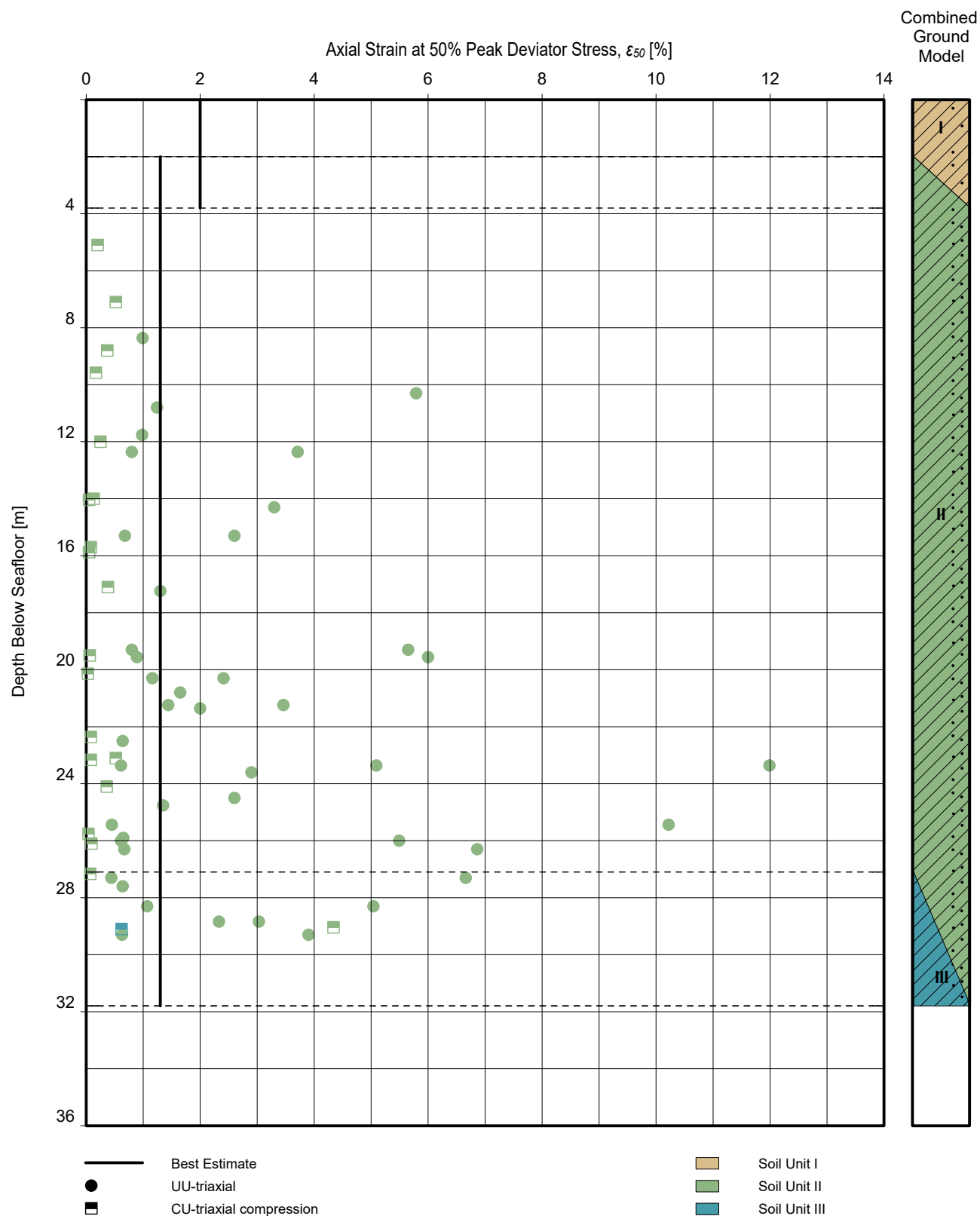


COEFFICIENT OF PERMEABILITY VERSUS DEPTH

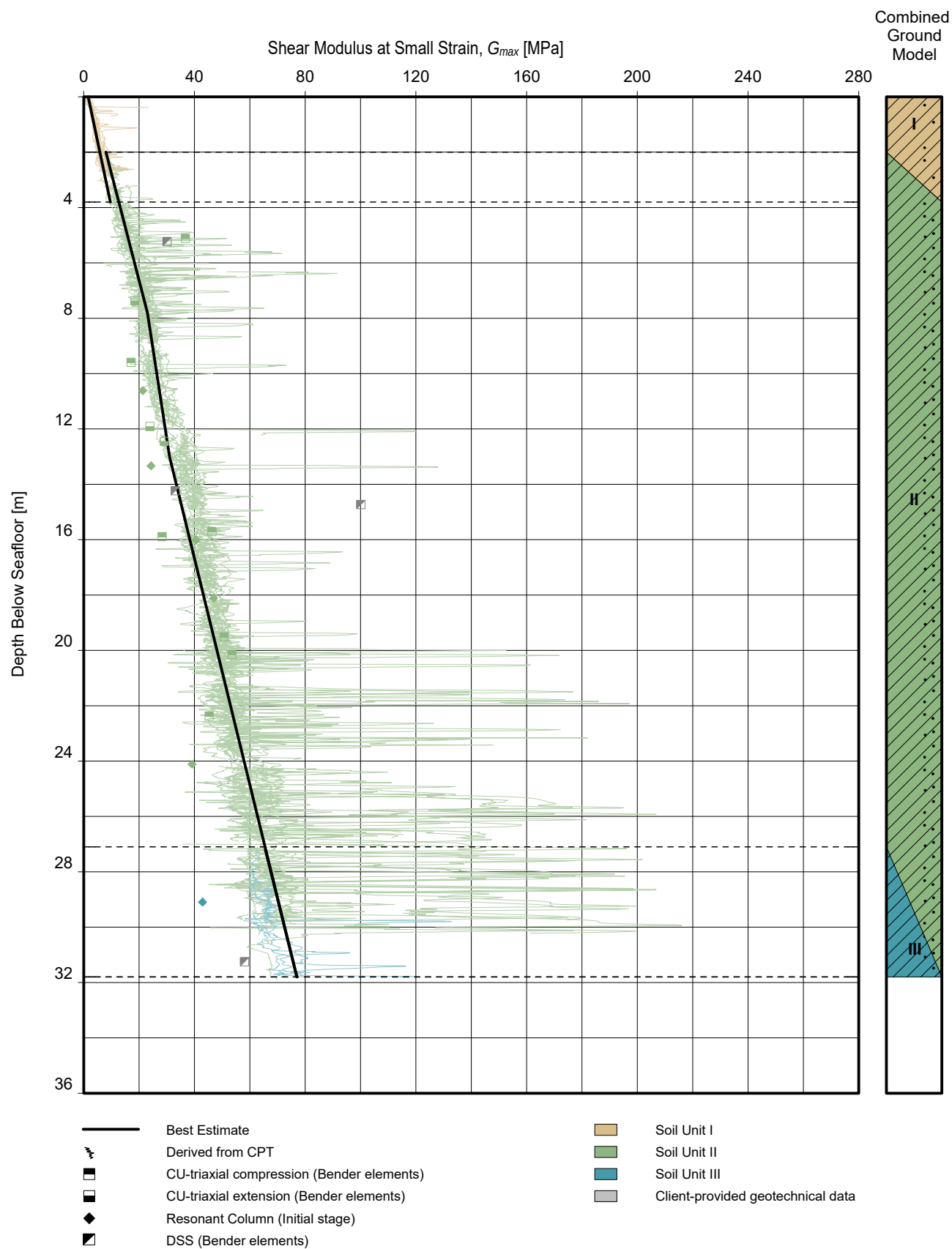


Note(s):
 - Refer to Chapter 3 for more information about units and locations represented

COEFFICIENT OF CONSOLIDATION VERSUS DEPTH

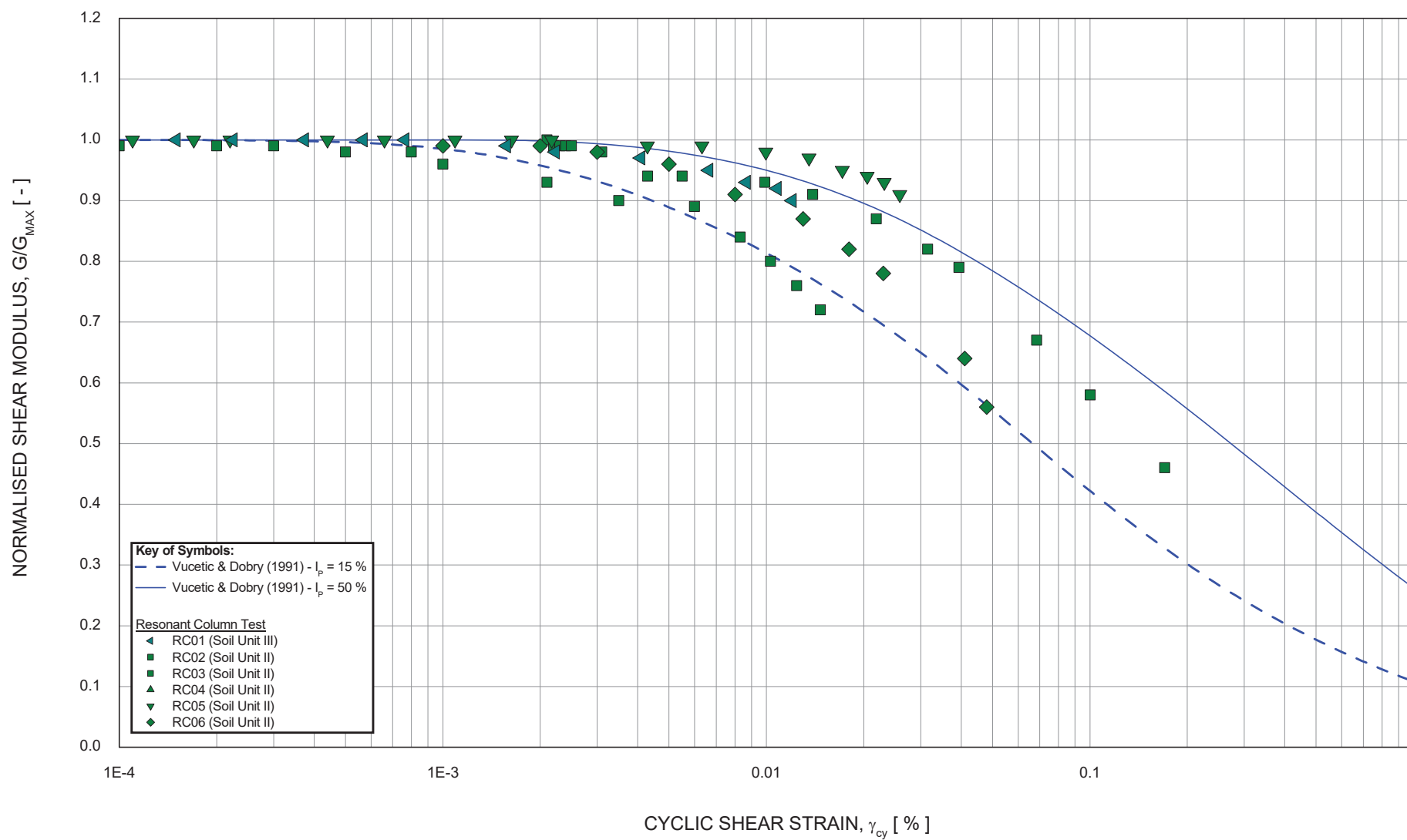


AXIAL STRAIN AT 50% PEAK DEVIATOR STRESS VERSUS DEPTH

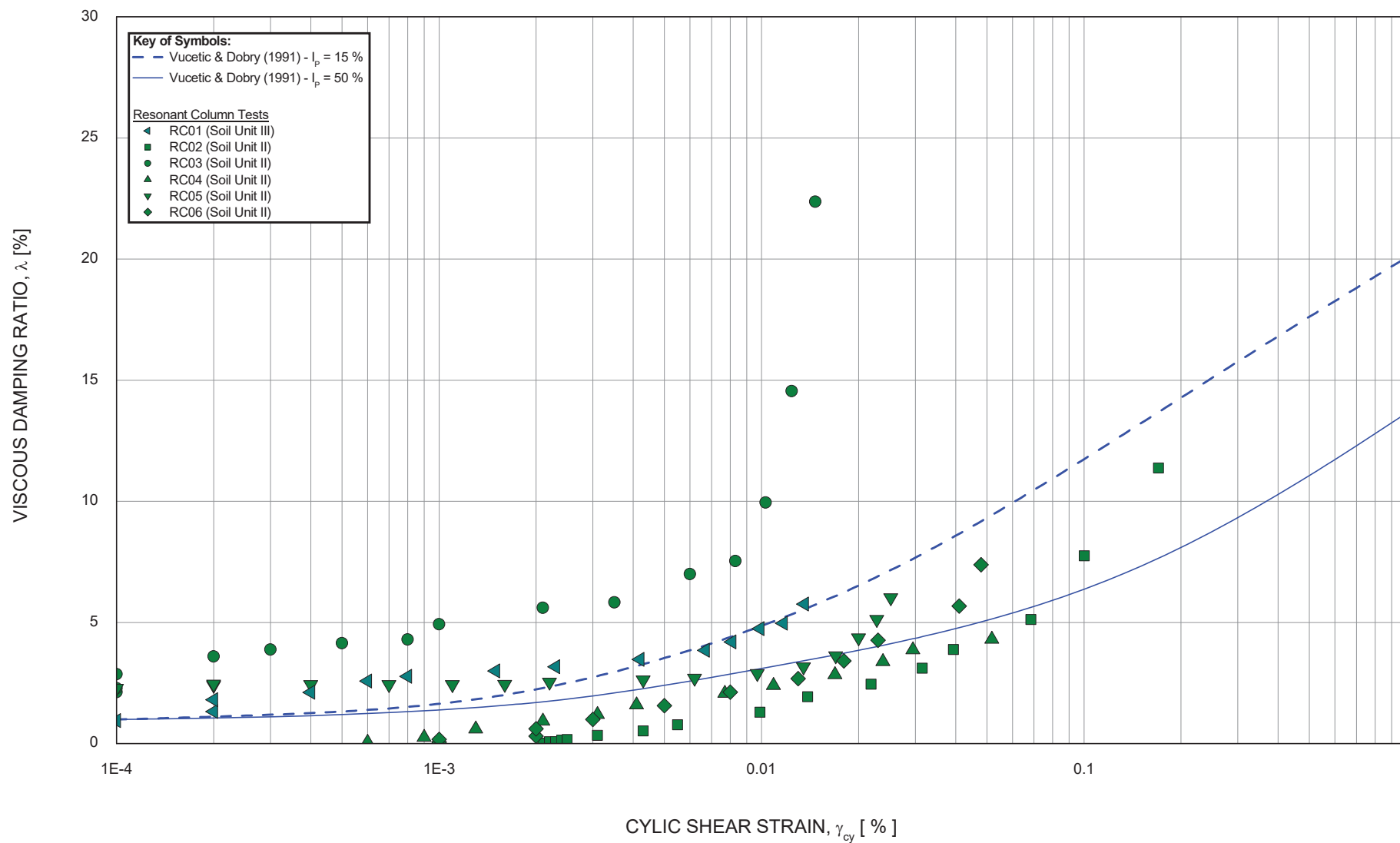


Note(s):
- Refer to Chapter 3 for more information about units and locations represented

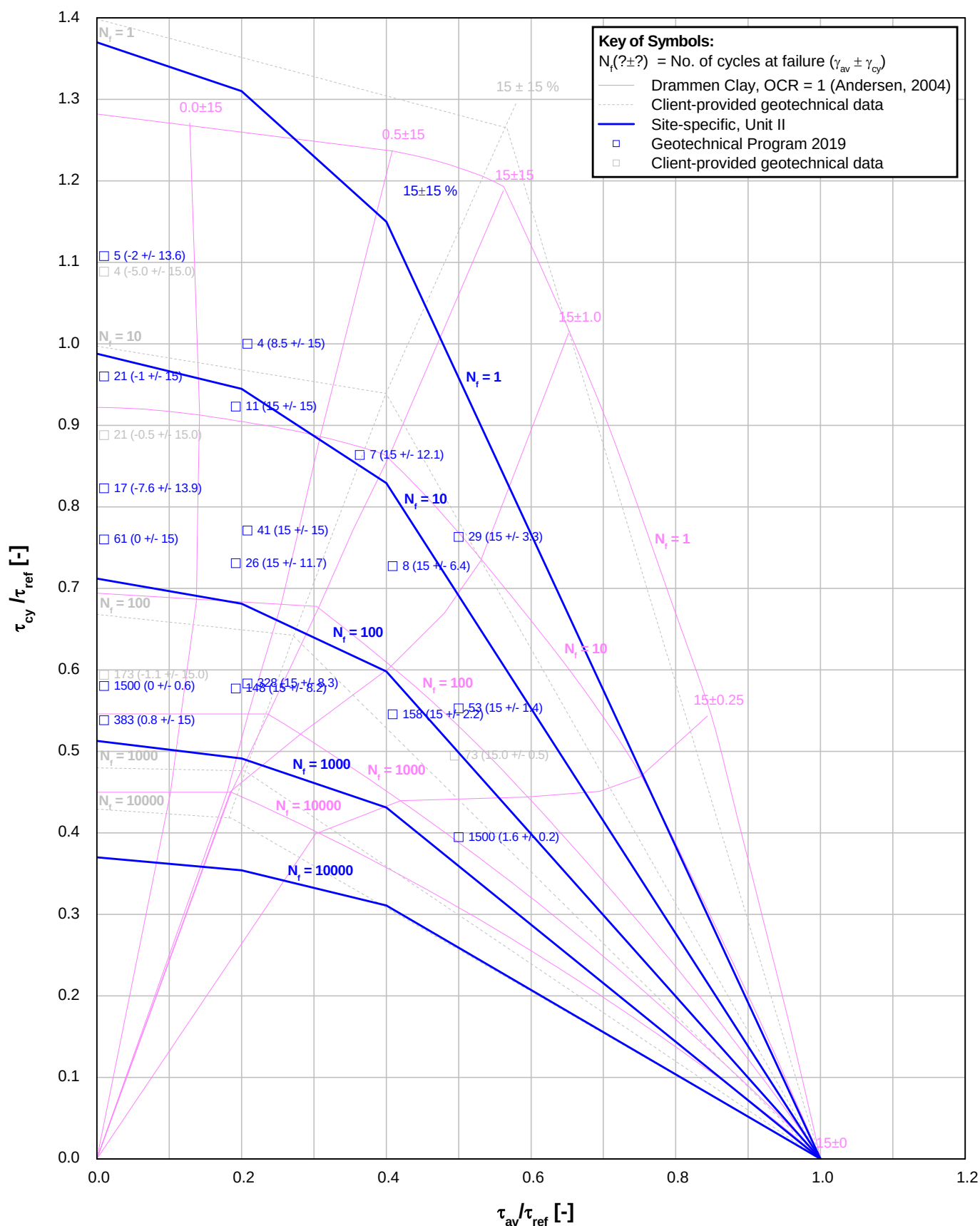
SHEAR MODULUS AT SMALL STRAIN VERSUS DEPTH



NORMALISED SHEAR MODULUS ($G/G_{\max}$) VERSUS CYCLIC SHEAR STRAIN (γ_{cy})

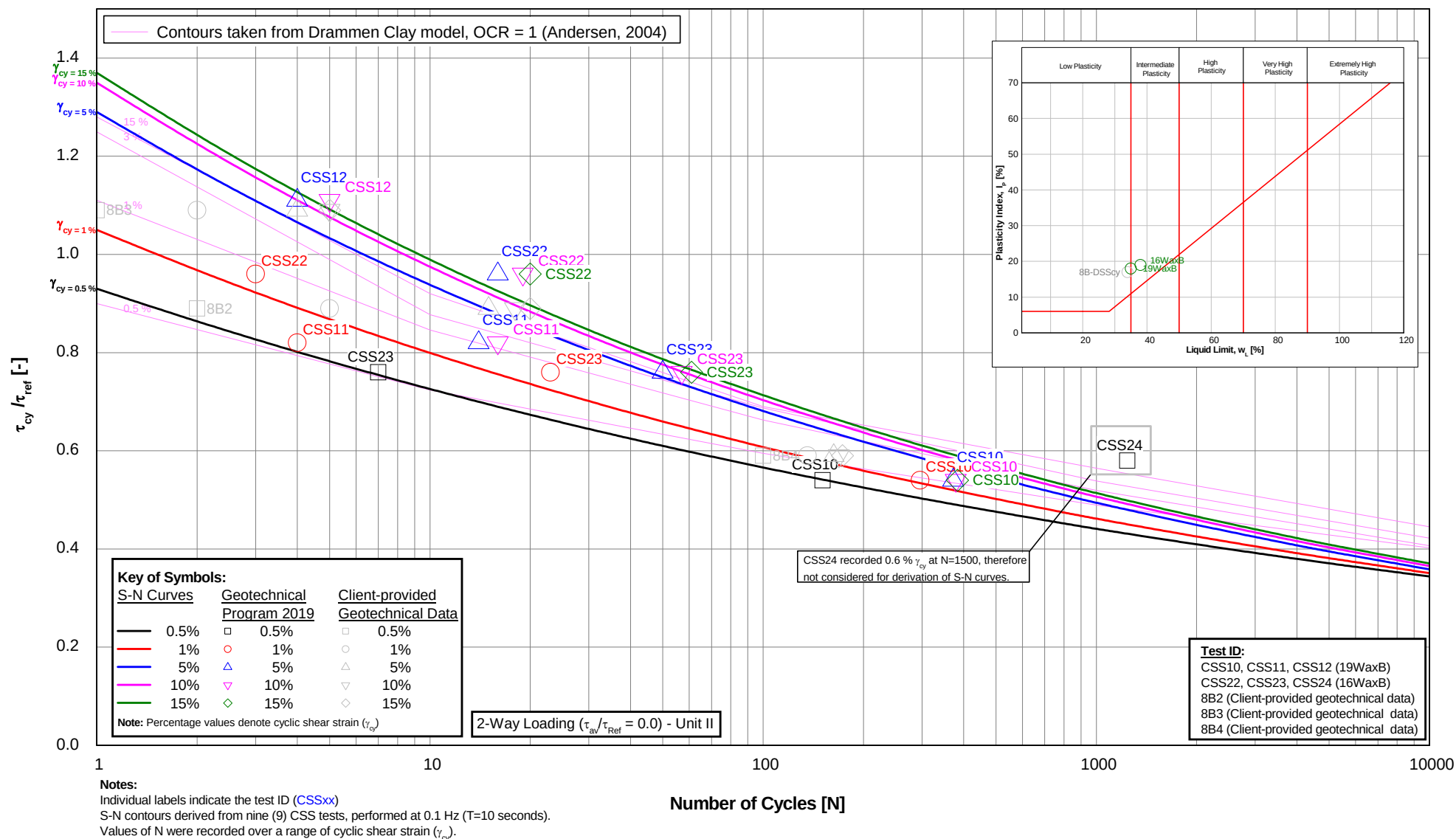


VISCOUS DAMPING RATIO (λ) VERSUS CYCLIC SHEAR STRAIN (γ_{cy})



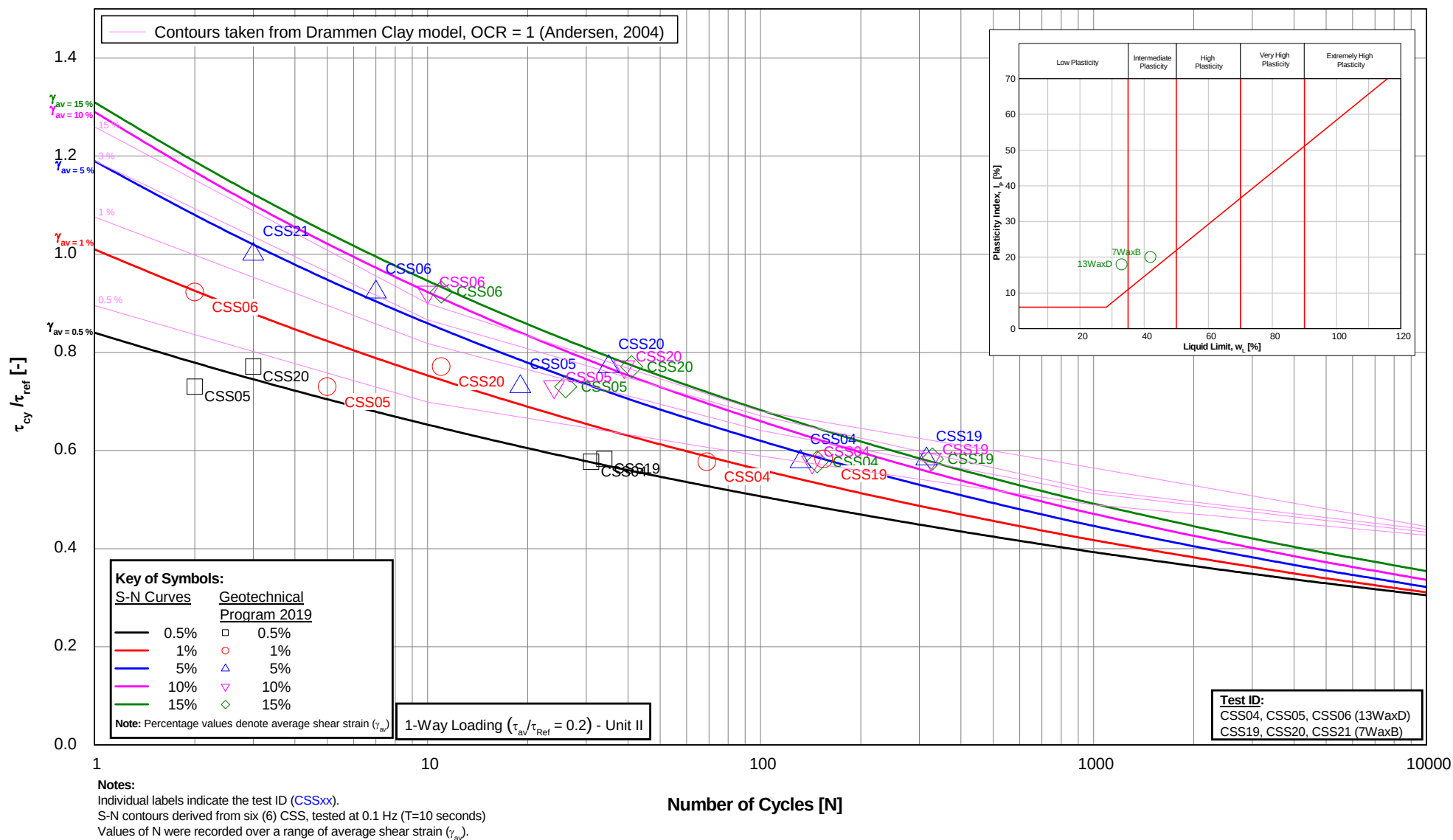
NORMALISED CYCLIC SHEAR STRESS VERSUS NORMALISED AVERAGE SHEAR STRESS

BAY DU NORD GP 2019 - ATLANTIC OCEAN, CANADA



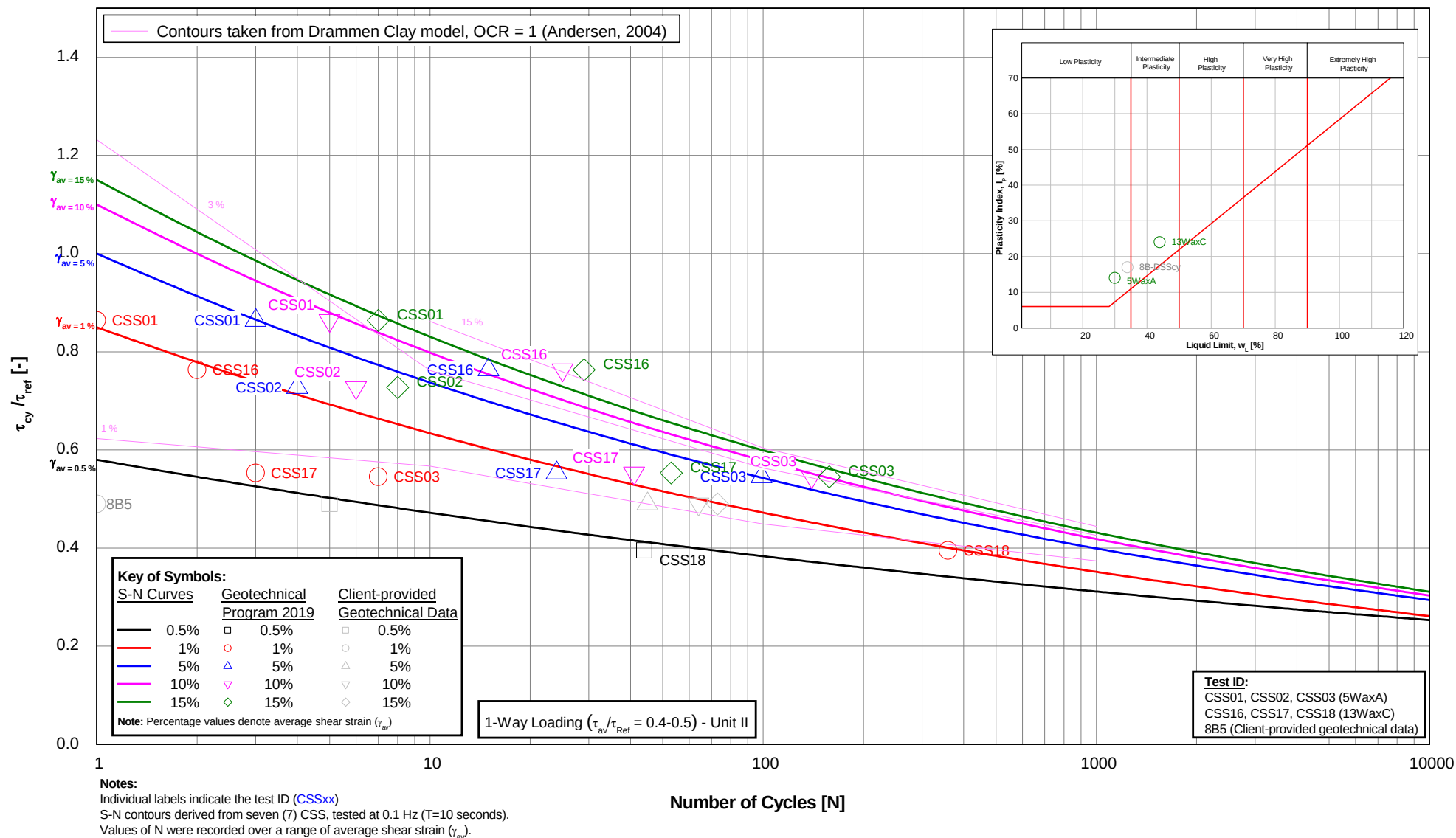
CYCLIC STRESS RATIO (S) VERSUS NUMBER OF CYCLES (N)

BAY DU NORD GP 2019 - ATLANTIC OCEAN, CANADA



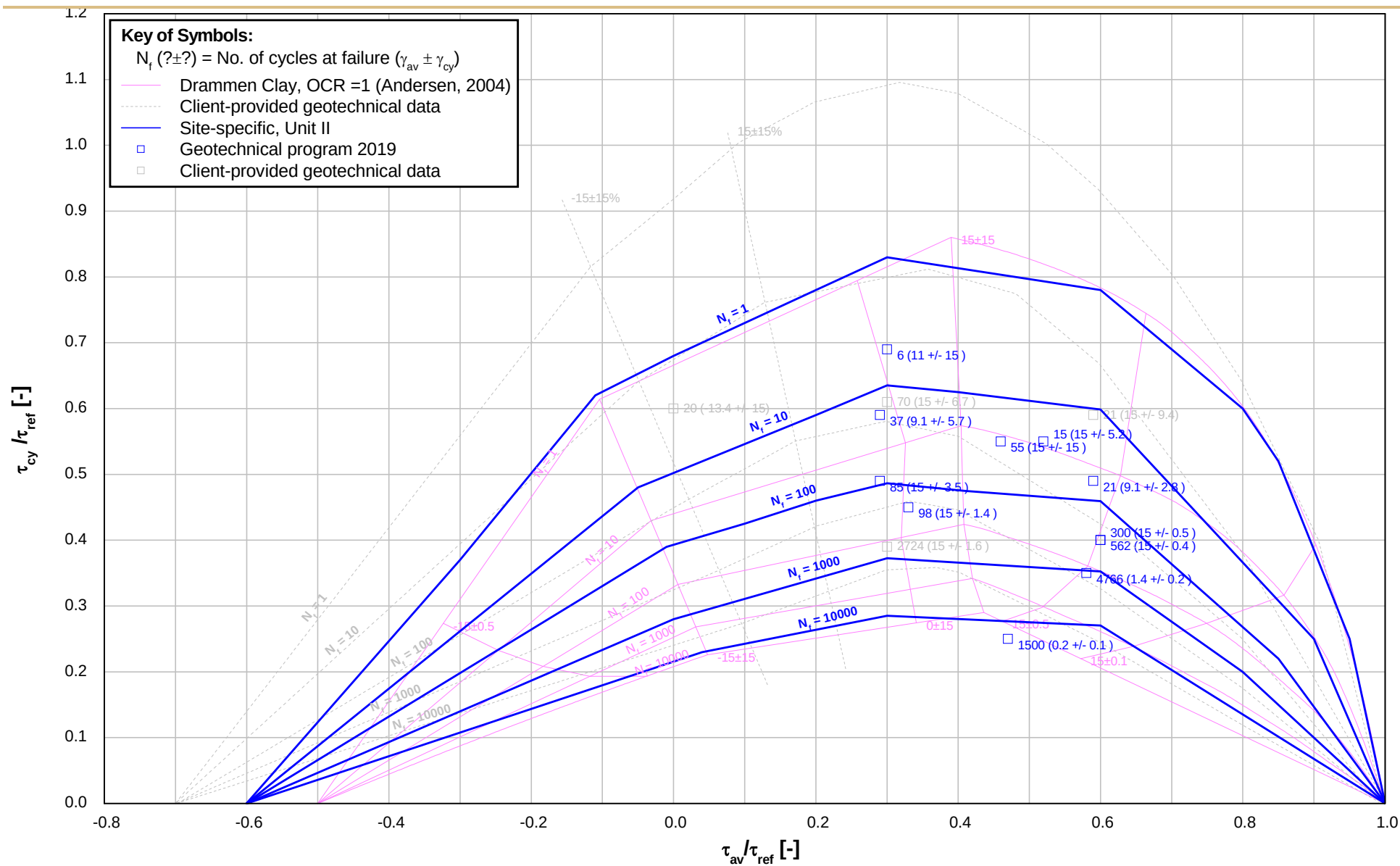
CYCLIC STRESS RATIO (S) VERSUS NUMBER OF CYCLES (N)

BAY DU NORD GP 2019 - ATLANTIC OCEAN, CANADA



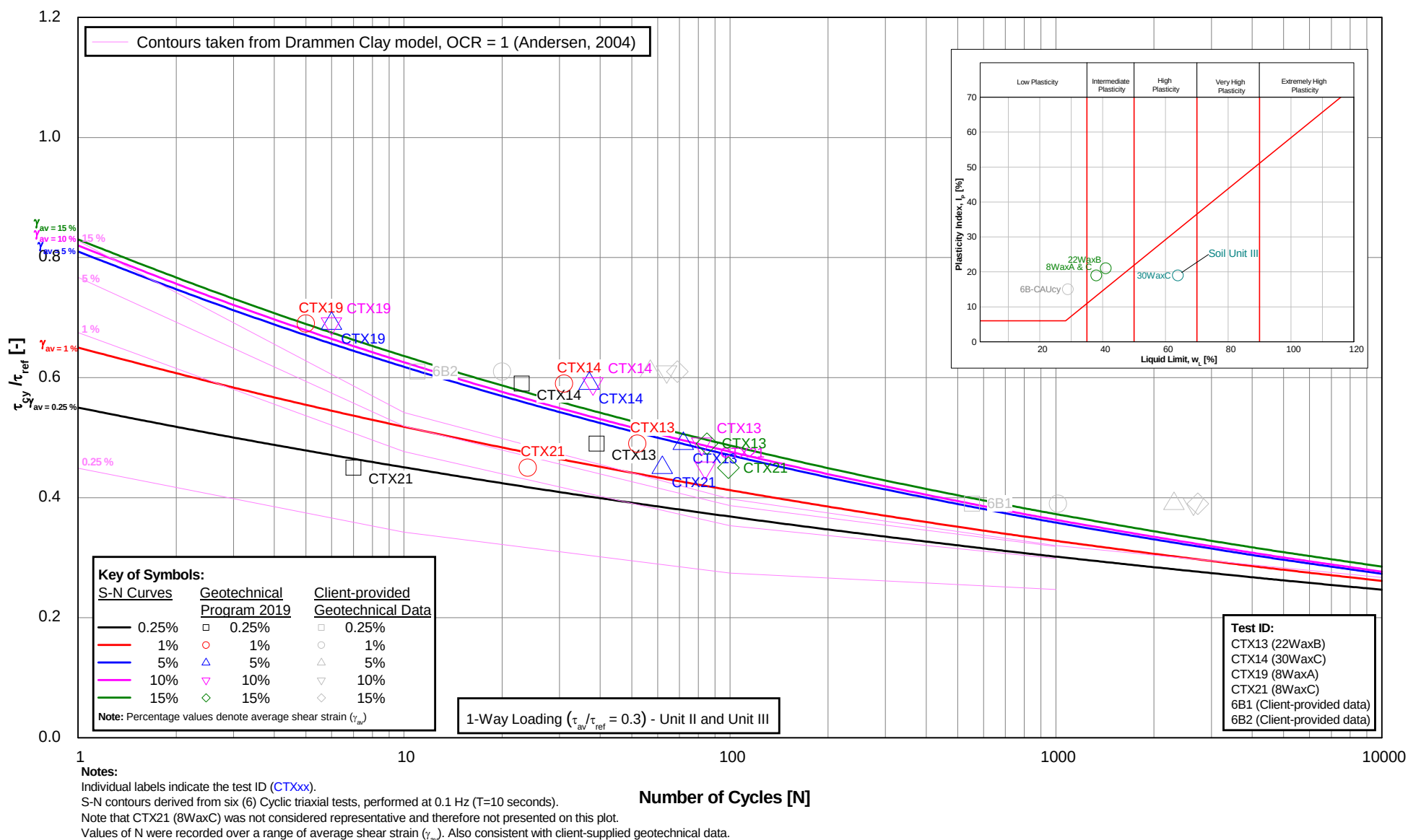
CYCLIC STRESS RATIO (S) VERSUS NUMBER OF CYCLES (N)

BAY DU NORD GP 2019 - ATLANTIC OCEAN, CANADA

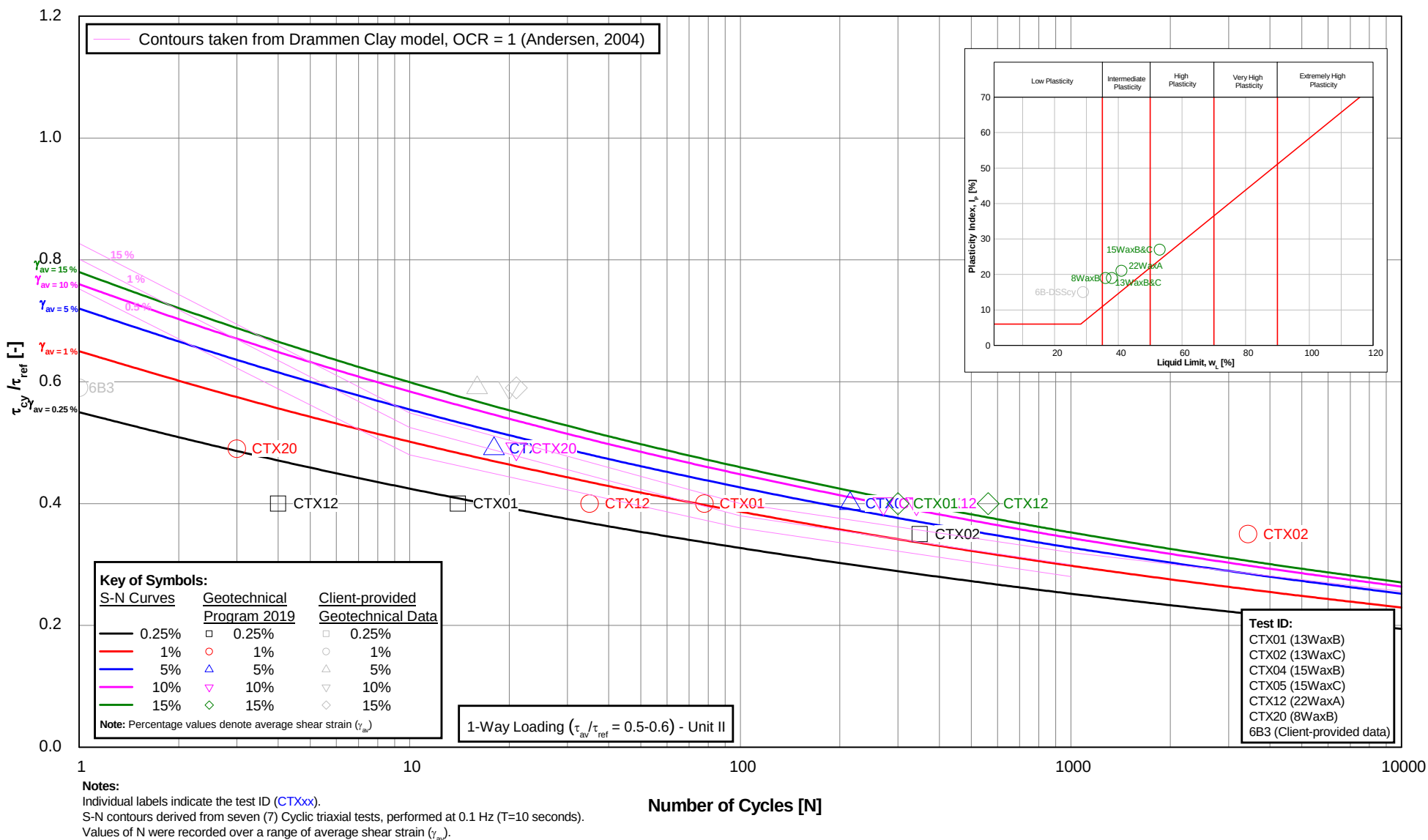


NORMALISED CYCLIC SHEAR STRESS VERSUS NORMALISED AVERAGE SHEAR STRESS

BAY DU NORD GP 2019 - ATLANTIC OCEAN, CANADA



CYCLIC STRESS RATIO (S) VERSUS NUMBER OF CYCLES (N)



CYCLIC STRESS RATIO (S) VERSUS NUMBER OF CYCLES (N)

CHAPTER 4 – RECOMMENDED PARAMETERS

4. RECOMMENDED PARAMETERS

4.1 Introduction

This chapter provides an overview, in tabular format, of LE, HE and/or BE profiles presented in Chapter 3. Note that this Chapter 4 presents no new information.

4.2 Results

Plate 4.2-1 presents a summary table of estimate profiles provided in Chapter 3. A colour scheme, as introduced in Chapter 3, is used to identify the different soil units.

In support of design, sub-site unit thicknesses are provided in Table 4.1 below.

Table 4.1: Sub-site unit thickness overview.

Sub-site	Location	Soil Unit Thickness [m]		
		I	II	III
Template North	EQ1945-BH-01(A)	2.8	25.8	>3.2
	EQ1945-BH-02			
	EQ19452-BH-03			
	EQ19452-BH-04			
Template Central	EQ19452-BH-11	3.2	26.6	>2.1
	EQ19452-BH-12			
	EQ19452-BH-13			
	EQ19452-BH-14			
Template South	EQ19452-BH-21	3.5	>28.3	N/A
	EQ19452-BH-22(A)			
	EQ19452-BH-23			
	EQ19452-BH-24			
Template Baccalieu	EQ19452-BH-31(A)	2.7	>27.3	N/A
	EQ19452-BH-32			
	EQ19452-BH-33			
	EQ19452-BH-34			
FPSO Anchor 1	EQ19452-BH-41(A)	2.9	>25.9	N/A
	EQ19452-BH-42(A)			
FPSO Anchor 2	EQ19452-BH-43	3.8	24.2	>2.0
	EQ19452-BH-44(A)			
FPSO Anchor 3	EQ19452-BH-45	2	25.1	>2.7
	EQ19452-BH-46			
FPSO Anchor 4	EQ19452-BH-49	2.1	26	>3.1
	EQ19452-BH-50(A)			
FPSO Anchor 5	EQ19452-BH-51	2.5	>27.3	N/A
	EQ19452-BH-52			
FPSO Anchor 6	EQ19452-BH-53	2.1	25.9	>2
	EQ19452-BH-54			
Notes: >xx.x = Minimum unit thickness confirmed during GP 2019 N/A = Not available; unit below maximum investigation depth of GP 2019				

List of Plates

	Plate
Summary Table of Geotechnical Parameter Estimate Profiles	4.2-1

BAY DU NORD GP 2019 – ATLANTIC OCEAN, CANADA

		In Situ Profiling		Basic Physical Properties		In Situ Stresses & Stress History	Static Undrained Shear Strength				
Unit	Depth [m]	q _n [kPa]		γ [kN/m ³]		OCR [-]	s _u [kPa]		s _{u,r} [kPa]		S _t [-]
		BE	SD	BE	SD	BE	LE	HE	LE	HE	BE
I	0.0	45	47	17.0	1.4	4.0	0.5	10.0	0.0	0.9	4.0
	2.0	90		17.0		1.5	4.2	13.7	1.3	2.4	4.0
	3.8	130		17.0		1.6	7.5	17.1	2.5	3.9	4.0
II	2.0	120	62	18.5	1.0	1.5	7.5	17.4	0.6	4.5	4.0
	3.8	207		18.5		1.6	13.8	24.1	1.1	5.0	4.0
	7.8	400		18.5		1.8	27.8	38.9	2.1	6.3	4.0
	13.0	490		18.5		1.3	34.0	47.6	3.4	7.8	4.0
	27.2	966		18.5		1.3	67.1	93.9	6.9	12.2	4.0
	31.8	1120		18.5		1.3	77.8	108.9	8.1	13.6	4.0
III	27.2	966	66	15.8	0.6	1.3	67.1	93.9	2.3	9.3	4.0
	31.8	1120		15.8		1.3	77.8	108.9	2.3	9.3	4.0

Stiffness, Permeability and Volume Change Behaviour									
Unit	Depth [m]	M [MPa]		k [m/s]		c_v [m ² /s]		ϵ_{50} [%]	G_{max} [MPa]
		LE	HE	LE	HE	LE	HE	BE	BE
I	0.0	0.0	0.6	1.0E-10	3.0E-09	3.1E-08	2.2E-06	2	1.5
	2.0	0.4	0.9	1.0E-10	3.0E-09	3.1E-08	2.2E-06	2	5.7
	3.8	0.6	1.2	1.0E-10	3.0E-09	3.1E-08	2.2E-06	2	9.5
II	2.0	0.6	1.5	1.0E-10	3.0E-09	3.1E-08	2.2E-06	1.3	8.0
	3.8	1.2	2.0	1.0E-10	3.0E-09	3.1E-08	2.2E-06	1.3	12.7
	7.8	2.3	3.2	1.0E-10	3.0E-09	3.1E-08	2.2E-06	1.3	23.0
	13.0	2.9	4.7	1.0E-10	3.0E-09	3.1E-08	2.2E-06	1.3	31.0
	27.2	5.6	8.1	1.0E-10	3.0E-09	3.1E-08	2.2E-06	1.3	65.7
	31.8	6.5	9.2	1.0E-10	3.0E-09	3.1E-08	2.2E-06	1.3	77.0
III	27.2	5.6	8.1	1.0E-10	3.0E-09	3.1E-08	2.2E-06	1.3	65.7
	31.8	6.5	9.2	1.0E-10	3.0E-09	3.1E-08	2.2E-06	1.3	77.0

Key:			k = Coefficient of permeability			BE = Best estimate		
q_n = Net cone resistance			c_v = Coefficient of consolidation			LE = Low estimate		
s_u = Undrained shear strength			ϵ_{50} = External axial strain at half the maximum deviator stress			HE = High estimate		
$s_{u,r}$ = Undrained shear strength of remoulded soil			γ = Unit weight			SD = Standard Deviation (of residuals)		
S_t = Sensitivity			M = Constrained modulus					
T_x = Thixotropy strength ratio			G_{max} = Shear modulus at small strain					
OCR = Overconsolidation ratio								
Note(s):								
- Values printed <i>italics</i> do not present inflection points or changes in trends. These values are linearly interpolated from adjacent data points.								
- Estimate lines are based on general soil conditions, i.e. clay.								

SUMMARY TABLE OF GEOTECHNICAL PARAMETER ESTIMATE PROFILES

CHAPTER 5 – REFERENCES

5. REFERENCES

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APPENDIX 1: DESCRIPTIONS OF METHODS AND PRACTICES**CONTENTS****Reference**

Use of Geodata and Advice	FNLM/GEO/APP/077
Positioning Survey and Depth Measurement	FNLM/GEO/APP/029
Geotechnical Borehole	FNLM/GEO/APP/002
Geotechnical Log	FNLM/GEO/APP/078
Soil Description and Classification	FNLM/GEO/APP/005
Geotechnical Laboratory Tests	FNLM/GEO/APP/007
Cyclic and Dynamic Laboratory Tests	FNLM/GEO/APP/051
Cone Penetration Test	FNLM/GEO/APP/001
Cone Penetration Test Interpretation	FNLM/GEO/APP/012
Marine Reflection Seismics	FNLM/GEO/APP/016
Site Characterisation	FNLM/GEO/APP/075
Geotechnical Analysis	FNLM/GEO/APP/052

This appendix presents method statements and terminology that are generally familiar to expert users of the information.

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 ISO 9001 quality principles for project management and ISO 2394 for general principles on reliability for structures. 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.

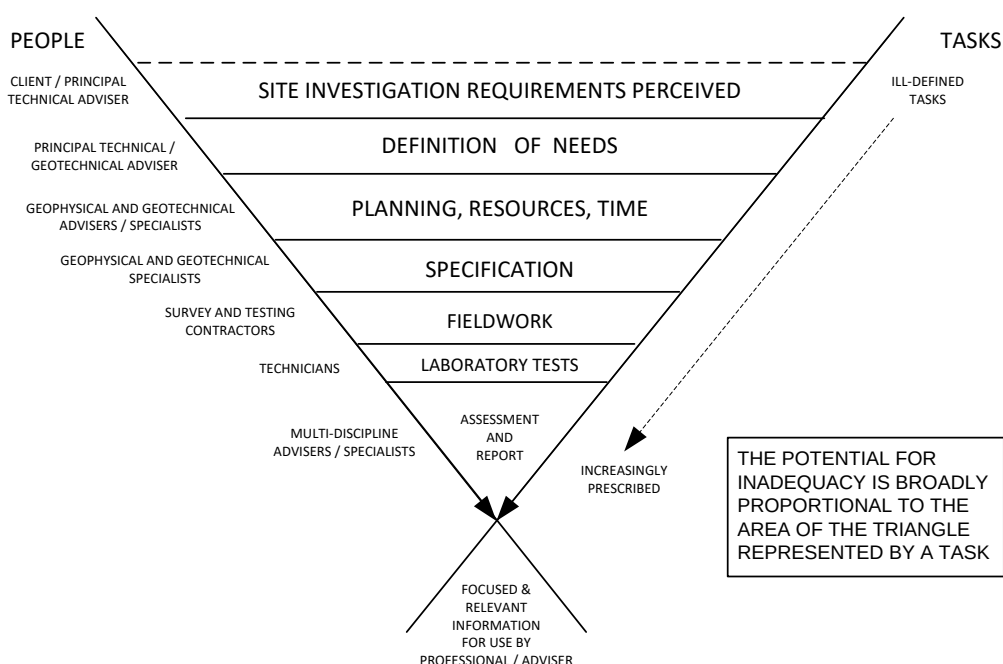


Figure 1: Quality of Site Investigation (adapted from SISG¹).

¹ Site Investigation Steering Group SISG 1993. *Site Investigation in Construction 2: Planning, Procurement and Quality Management*. London: Thomas Telford.

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 Limitations

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, and users of geodata and advice should confer with the authors when applying the geodata and/or advice.

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. In general, two years after the date of geodata, analyses and advice, the information may be considered inaccurate or unreliable. A specialist should be consulted regarding the adequacy of the geodata, analyses and advice for use after any passage of time.

Positioning Survey and Depth Measurements

Introduction

This document describes survey of horizontal and elevation/depth reference points for geotechnical and/or environmental data acquisition in a marine environment.

National and international standards for geotechnical and/or environmental data acquisition (as ASTM, BSI, CEN and ISO) require such surveys, but do not describe procedural details. 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 subsurface 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;
- 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;

- subsurface positioning survey, i.e. adjustment of the surface positioning results for underwater offset;
- measurement of the water depth;
- calculation of elevation of seafloor or a data point for the seabed relative to a vertical datum, e.g. water level correction.

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.

Survey Classifications

Positioning surveys require specific systems and procedures, such as those summarised below for marine applications. The International Hydrographic Organization (IHO, 2008) defines four orders of hydrographic survey (Table 1). The term "depth" refers here to water depth, i.e. the vertical distance between water level and seafloor. Water level can be expressed with reference to, for example, Lowest Astronomical Tide LAT as vertical datum. Note that a water level such as LAT depends on the calculation model used for determining the vertical datum.

Table 1: Summary of IHO Classification

IHO Order	Special	1a	1b	2
Description of Areas	Areas where under-keel clearance is critical	Areas shallower than 100 m where underkeel clearance is less critical but features of concern to surface shipping may exist	Areas shallower than 100 m where under-keel clearance is not considered to be an issue for the type of surface shipping expected to transit the area	Areas generally deeper than 100 m where a general description of the seafloor is considered adequate
Maximum Allowable Total Horizontal Uncertainty 95 % Confidence Level	2 m	5 m + 5 % of depth	5 m + 5 % of depth	20 m + 10 % of depth
Maximum Allowable Total Vertical Uncertainty 95 % Confidence Level	a = 0.25 m b = 0.0075	a = 0.5 m b = 0.013	a = 0.5 m b = 0.013	a = 1.0 m b = 0.023
Full Seafloor Search	Required	Required	Not required	Not required
Feature Detection	Cubic features > 1 m	Cubic features > 2 m in depths up to 40 m; 10 % of depth beyond 40 m	Not applicable	Not applicable
Recommended Maximum Line Spacing	Not defined as full seafloor search is required	Not defined as full seafloor search is required	3 x average depth or 25 m, whichever is greater	4 x average depth

Note: The use of coefficients a and b is as follows:

$$\pm\sqrt{a^2 + (b * d)^2}$$

where:

- a represents that portion of the uncertainty that does not vary with water depth
- b is a coefficient which represents that portion of the uncertainty that varies with water depth
- d is the water depth
- b * d represents that portion of the uncertainty that varies with water depth.

Figure 1 illustrates the effect of coefficients a and b.

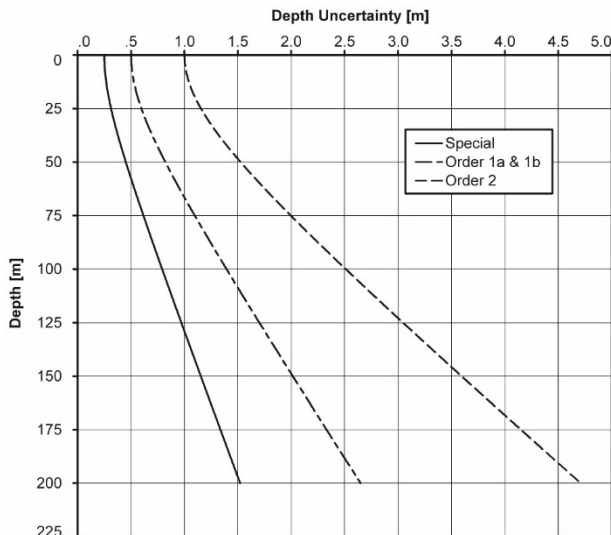


Figure 1: IHO water depth uncertainty

IHO Survey Classification - Offshore Practice Examples

The IHO Special Order Survey is exceptional in geotechnical and/or environmental data acquisition. A Special Order system set-up may be comprised of: RTK DGPS; a multibeam echosounder; a motion compensator, and a conductivity temperature depth (CTD) probe. Subsurface 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) subsurface 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) subsurface 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). For example, surface position may be determined by two independent DGPS systems or direct sounding by drill pipe and echosounding.

Comments on Uncertainty Budget

IHO Order and offshore system set-ups involve relatively complex uncertainty budgets (uncertainty estimates). IHO considers total propagated uncertainties for the reference point on the seafloor. For example, horizontal positioning must not only consider the uncertainty of a DGPS antenna position, but also uncertainty in offset between antenna and actual position of a tool on the seafloor.

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 m.
- RTK DGPS – antenna position uncertainty typically in the order of centimetres.
- Gyro compass – uncertainty typically in the order of 0.5° to 1°.

- DGPS uncertainty contributions include the geodetic network, vessel dynamics and antenna offset. Continuous logging on location allows some quantification of position uncertainty.

Subsurface 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, and 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.

Uncertainty budgets can be project-specific. Soft soils, for example, can introduce uncertainty in underwater vertical position of measurement. 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 soil may 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".

Depth Below Seafloor

This section describes estimation of 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) provides depth accuracy classes, as shown in Table 2. 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.

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

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.

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
- 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

Characteristics	Marine Setting	
	Favourable	Adverse
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

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 sediments.

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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™.

Optional activities for rotary drilling are MWD (measuring while drilling) and LWD (logging while drilling). MWD involves the manual or digital recording of drilling parameters, such as penetration rate, torque and drill fluid pressure. LWD involves sensors located in a drill collar immediately above the drill bit. The downhole sensors can include electrical resistivity, natural gamma ray, sonic velocity and temperature.

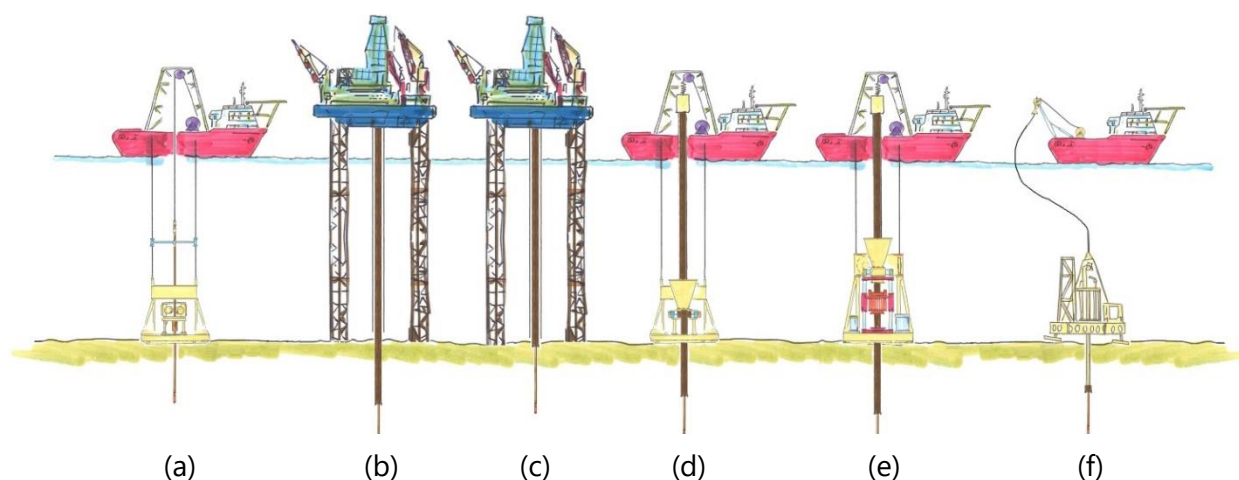


Figure 1: Deployment modes for marine soil investigation, in combination with cone penetration testing

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), thermal conductivity test (TCT) 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.
- 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.

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:

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

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:

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

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.

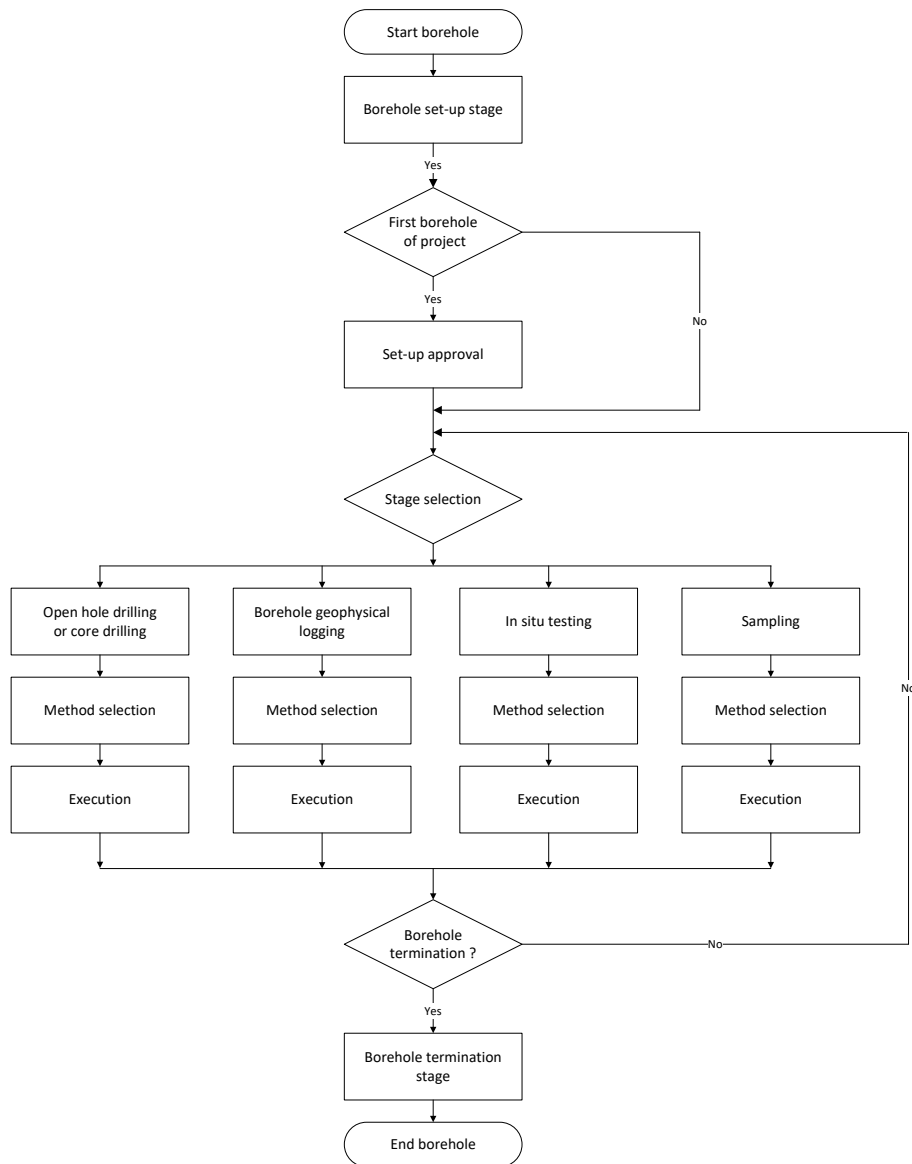


Figure 2: Flow chart

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, MWD and LWD
- 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 parameters
- selection of subsequent drilling stage.

In Situ Testing

- in situ test
- logging, such as test depth and test parameters
- selection of subsequent drilling stage.

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 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.

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).

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.						

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.		

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.

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 oxidation 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.

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 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.

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.

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 logging, sample or test data are presented.

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.

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.

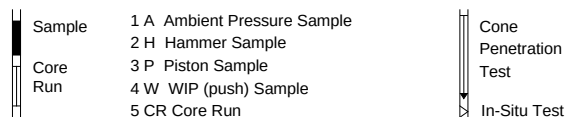


Figure 1: Symbols for identification of samples and in situ tests

Drilling Parameters

Measurement while drilling (MWD) 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) and Robertson (2010). 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.

MAIN SOIL TYPE	CEMENTATION	ADDITIONAL SOIL PARTICLES	EXAMPLES OF GRAPHIC LOG
PEAT	Slightly Cemented	Organic Matter	PEAT, clayey
CLAY	Moderately Cemented	Shells or Shell Fragments	CLAY, sandy
SILT	Well Cemented	Coral Fragments	CLAY, very sandy
SAND		Algal Crustations	SILT, sandy
GRAVEL		Gypsum Crystals	SAND, clayey
Coralline DEBRIS		Rock Fragments	SAND, very gravelly, clayey
DEBRIS		Inclusions	SAND, silty, well cemented
Shell DEBRIS		Interbedded Thin Layer/Seam	GRAVEL, sandy
MADE GROUND			

Figure 2: Symbols for soils

CARBONATE ROCKS	SILICA ROCKS	EVAPORITES
 CALCARENITE	 SANDSTONE	 GYPSUM / ANHYDRITE
 CALCISILTITE	 SILTSTONE	 ROCK SALT
 CALCILUTITE	 CLAYSTONE	
 Carbonate CONGLOMERATE	 CONGLOMERATE	
 Carbonate BRECCIA	 BRECCIA	
 LIMESTONE	 MUDSTONE	
 Dolomitic LIMESTONE	 SHALE	
 CHALK		

Figure 3: Symbols for sedimentary rocks

METAMORPHIC ROCKS	IGNEOUS ROCKS
 Fine grained rock	 Fine grained rock
 Medium grained rock	 Medium grained rock
 Coarse grained rock	 Coarse grained rock

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.

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Soil Description and Classification

Introduction

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 (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 – Unified 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 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. Other differences include suggested format and order of soil description.

Where applicable, carbonate soil classification system can be integrated, using the system by Clark and Walker (1977). The characteristics of carbonate 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.

This document applies to soil description and classification for:

- sample description;
- soil specimen description, whereby a specimen can be all or part of sample;
- soil stratum description, for example in the format of a geotechnical log.

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.

The following sections summarise typical steps of BS 5930:2015, ISO 14688-1+2, and ASTM D2487-17/ASTM D2488-17e1, particularly:

1. Measure or estimate particle type as silica-based, organic, or calcareous.
2. For soils that are predominantly silica-based and/or organic, select BS or ASTM based on local geotechnical practice or project requirements, and follow the appropriate descriptive procedure; for calcareous soils, use the process described by Clark and Walker (1977).
3. Measure or estimate the particle size distribution and Atterberg limits for use in defining the principal and secondary soil fractions.
4. 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.
5. Complete the description and classification using the additional terms for the soil mass characteristics and other features such as bedding, colour, and particle shape.

Calcareous Soil Description

The procedure considers particle size, carbonate content and material strength. The particle size classification follows the Unified Soil Classification System. The carbonate content is an additional feature and the material strength classification relates to common post-depositional alteration of calcareous soil.

Particle Type

The first determinant for soil description is particle type using Table 1 It mainly differentiates between silica and carbonate soil compositions with organic content of less than 1 % of the dry weight. Organic soils are further described in the soil description procedures for BS and ASTM.

Table 1: Particle Type

Fine Soils	Coarse Soils	Carbonate Content (by dry weight)	Reaction with HCl (10 %)
--	Silica	< 10 %	In fine soils: no bubbles, or slowly forming bubbles. In coarse soils: reaction often limited to some individual particles, or particle surface Residue: nearly all soil remaining
Calcareous	Calcareous silica	10 to 50	In fine soils: clearly visible, prolonged reaction and foaming. In coarse soils: violent reaction Residue: large part of soil remaining
Carbonate	Siliceous carbonate	50 to 90	Violent reaction Residue: only small part of soil remaining
Carbonate	Carbonate	> 90	Violent reaction Residue: hardly any soil remaining

The description method does not distinguish between types of carbonate material, and assumes that non-carbonate particles are siliceous.

Cementation and Lithification

Cementation is the process by which a binding material precipitates in the voids between the grains or minerals. Lithification is the process by which a soil is hardened due to pressure solution and transformation or new grain or mineral growth. Both processes contribute to the formation of rock.

The descriptions for cementation follow rock strength classification (Table 2) expressed as uniaxial compressive strength σ_c :

Table 3: Lithification

Carbonate Content [%]	Dominant Fraction						σ _c
	Clay	Silt	Sand	Gravel	Cobbles	Boulders	[MPa]
Incomplete Lithification							
< 10	CLAYSTONE	SILTSTONE	SANDSTONE	CONGLOMERATE	CONGLOMERATE or BRECCIA	0.3 to 12.5	
10 to 50	Calcareous CLAYSTONE	Calcareous SILTSTONE	Calcareous SANDSTONE	Calcareous CONGLOMERATE			
50 to 90	Clayey CALCILUTITE	Siliceous CALCISILTITE	Siliceous CALCARENITE	Conglomeratic CALCIRUDITE			
> 90	CALCILUTITE	CALCISILTITE	CALCARENITE	CALCIRUDITE			
Complete Lithification							
< 50	CLAYSTONE	SILTSTONE	SANDSTONE	GRAVEL CONGLOMERATE	CONGLOMERATE or BRECCIA	> 12.5	
	Fine-grained argillaceous LIMESTONE	Fine-grained siliceous LIMESTONE	Medium-grained LIMESTONE	Conglomeratic LIMESTONE			

The Clark and 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.

Soil Description and Classification using BS 5930:2015 and ISO 14688-1+2

Soil Group

The soil group subdivides the soils into very coarse, coarse, fine, and 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 weight percentage.

Organic soils contain usually small quantities of dispersed organic matter that can have a significant effect on soil plasticity

Table 2: 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 2 applies to soil, which also shows sublayers with little or no cementation. In case of further lithification, the soil description becomes a rock description using Table 3. The rock strength is only indicative.

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 constituent in inorganic soil, the terms in Table 4 are used.

Table 4: Terms for Description of Secondary Organic Matter in an Inorganic Soil

Term	Typical Colour	Organic Content	Organic Content [weight % of dry mass]
Slightly organic	Grey	Low organic content	2 to 6
Organic	Dark grey	Medium organic content	6 to 30
Very organic	Black	High organic content	> 30

Soils with organic contents of up to approximately 30 % by weight and water contents of up to about 250 % behave as inorganic, mineral soils and are described using the terms given in the lower portion of Table 4. 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/m³ to 1.1 Mg/m³. 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).

Principal Soil Type

Classification of coarse and 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 remould, it is described as a coarse soil ("SAND" or "GRAVEL" dependent on its particle size distribution). The principal soil type which dominates the soil behaviour and the secondary constituents which modify that behaviour is described.

Coarse Soils

The principal soil type 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 5.

Table 5: Grading Characteristics from Particle Size Distribution

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 6. 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" allow differentiation between predominant fractions and a range of sizes.

Table 6: Size Fraction Descriptions for Coarse Soils

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. The description of plasticity on-site can be carried out using the terms low plasticity or high plasticity, based on hand tests. In a laboratory, plasticity can be determined based on the results of Atterberg limits tests (Table 7).

Table 7: Plasticity Classification

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 to be 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 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.

Fine soils are described as either a "SILT" or a "CLAY" depending on the plastic properties. On-site distinction between SILT and CLAY can be made using hand tests. 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 (Table 8). The terms in Table 8 can be used, where appropriate.

Table 8: Terms for Particle Shape

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

Secondary Constituents

Table 9 presents terms for ranges of secondary constituents. 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.

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 9: Terms for Mixtures of Fine and Coarse Soils

Term	Principal Soil Type	Approximate Proportion of Secondary Constituent	
		Coarse Soils	Fine Soils
-	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:			
* = Can be described as fine soil depending on soil behaviour			
† = Can be silty CLAY or clayey SILT			
‡ = Can be described as coarse soil depending on assessed soil behaviour			

Soil Colour

Soil colours are described using a Munsell soil colour chart (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.

Bedding and Interbedding

Layers of different soil types within a stratum are called bedding units (Table 10). 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 10, 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 10 and Table 11 respectively), e.g. "SAND with closely spaced thick laminae of clay". Where two or more soils 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. There 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 10: Bedding and Interbedding Thickness Terms

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 2000

Term	Thickness of Bedding Unit [mm]
Very thickly (inter)bedded [‡] With [†] very thick beds [‡]	> 2000
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 ranges from extremely closely spaced (< 20 mm) to very widely spaced (> 2000 mm). No other descriptive terms are used. An example would be: "firm grey very closely fissured fine sandy calcareous CLAY with many silt partings".

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 11: Spacing of Discontinuities

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 2000
Very widely spaced	> 2000

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 (CPT), 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 12. These ranges are in common use in the industry. They were originally presented in Lambe and Whitman (1979).

Table 12: 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 13, particularly if no information is available for description of undrained shear strength. When undrained shear strength data are available, terms can be refined to those given in Table 14.

Table 13: Consistency of Fine Soils

On-site 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 14: Undrained Shear Strength of Fine Soils

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

Tertiary Constituents

Tertiary constituents 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 trace", "with few", "with" and "with many" (in increasing order). These terms are usually added at the end of the primary soil description (e.g. "with many shell fragments", "with silt pockets"). These terms are qualitative and no percentage is given. The size of the tertiary constituents can be given in mm.

Mineral constituents are generally reported before the principal soil type, 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.

Soil Odour

For anthropogenic soils, odour can be described. Terms used to describe the odour are for example "H₂S", "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 sample description, and strong dependence on a person's ability to detect and identify odour.

Soil Description and Classification using ASTM D2487 and 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 3 inch 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 $\leq 12\%$ 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 to be 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 also sub-divided into coarse, medium, and fine, as defined in Table 15.

Table 15: 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 $\geq 75\%$ 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 $\geq 75\%$ of the liquid limit of the original specimen determined before oven drying).

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 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 16: 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

Table 17: Terms for 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 chart (e.g. Gretag-Macbeth, 2000).

Structure

Criteria for describing soil structure are provided in Table 18, along with additional terms in use in the geotechnical industry.

Table 18: 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 planes 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
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

Note: * = not part of ASTM D2488

The distance between the fissures, shear planes and expansion cracks is noted using the terms in Table 11.

Secondary Constituents

Table 17 presents a summary of terms to be used for ranges of secondary constituents. Applicable group symbols are defined in the previous section.

Consistency and Undrained Shear Strength of Fine-Grained Soils

The consistency of fine soils can be assessed on site, in case of no on-site laboratory. Terms to be used for the designation of consistency of fine-grained soils in accordance with the results of manual tests are given in Table 19.

Table 19: 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 20 can be presented.

Table 20: 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 the “Very hard” range have been added

Written Soil Descriptions

Soil description is typically given in consistent sequence, such as:

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 constituents either before or after the principal soil type as appropriate
7. PRINCIPAL SOIL TYPE, based on grading and plasticity.

For example: Firm closely fissured dark olive grey sandy calcareous CLAY with few silt pockets.

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 consists 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.

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 anhydrite 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

Testing Practice

General

This document summarises geotechnical laboratory test methods for soil.

Fugro strives to arrange testing in registered laboratories with formal accreditation. This document summarises test methods used by many Fugro geotechnical laboratories. Test methods used by on-site 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 (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.

Test standards typically provide explicit and indirect indications of applicability of a test method. An example of an explicit indication of applicability would be that the test is intended for uniform homogeneous clay. Indirect indications can consist of e.g. a low-stress limit for equipment capability and a general reference to public literature. In practice, laboratory tests may be performed outside of the intended applicability of the test method. Typically, such applicability situations are not explicitly reported.

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.

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. Usually, a reasonable estimate of test feasibility is possible at the time of sampling. A further refinement is possible in the laboratory prior to testing and, in some cases, only after testing. The limitations of feasibility estimates may lead to rejection of samples for testing upon inspection in the laboratory or may result in appropriate comments on test results after completion of testing.

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.

Sample Quality

The adopted classification system for sample quality is according to BSI (2015) and ISO (2006, 2014). The classification system recognises 5 sample quality classes or application classes, based

on feasibility of specific geotechnical identification and laboratory tests. A summary of these classes is as follows:

- Class 1: undisturbed: strength, stiffness and consolidation
 - Class 2: undisturbed: permeability, unit weight, boundaries of strata - fine
 - Class 3: disturbed: water content
 - Class 4: disturbed: particle size analysis, Atterberg limits, boundaries of strata - broad
 - Class 5: disturbed: sequence of layers
- The higher class includes the laboratory tests of the lower class.

An indication of intact (undisturbed) sample quality may be obtained from re-compression of a test specimen, for example in oedometer, direct simple shear or triaxial apparatus. Table 1 presents a method recommended by ISO (2014) based on $\Delta e/e_0$. Here, Δe represents the change in void ratio Δe from an initial laboratory value (e_0) 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: 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

The presented criteria are based on tests on marine clays 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 2 presents an alternative indication of intact (undisturbed) sample quality according to Terzaghi et al. (1996). Here, volumetric strain is derived from an initial laboratory specimen volume and the specimen volume upon re-compression to estimated in situ stress conditions. 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

The $\Delta e/e_0$ and ε_v 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 $\Delta e/e_0$ and ε_v .

The $\Delta e/e_0$ and ε_v 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 $\Delta e/e_0$ and ε_v should exclude secondary consolidation. In practice, no correction for secondary consolidation will be applied. This practice underestimates undisturbed sample quality, particularly for incremental loading oedometer tests with 24 hour load increments and longer.

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 D2487-17, ASTM D2488-17e1, BS 5930:2015, 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 %).

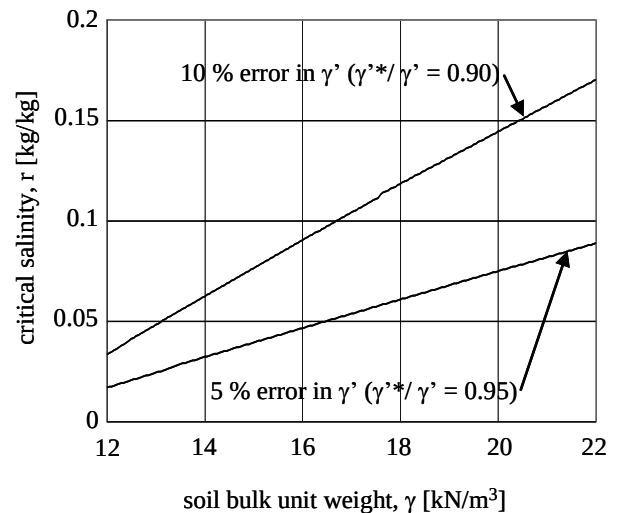
Test references: ASTM D2216-19, BS 1377-2:1990, ISO 17892-1:2014, ISO 19901-8:2014

Unit Weight – Volume-Mass Calculation

Measurement of volume and mass of a soil sample allows calculation of unit weight (γ). For fine-grained (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-grained (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 fine-grained specimens 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. The diagram below 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.

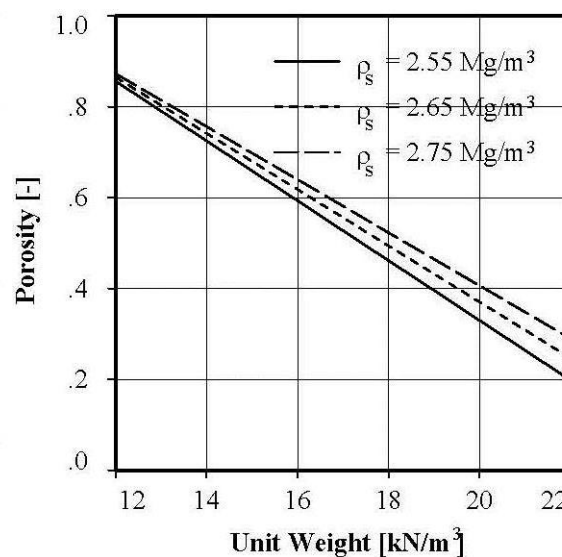
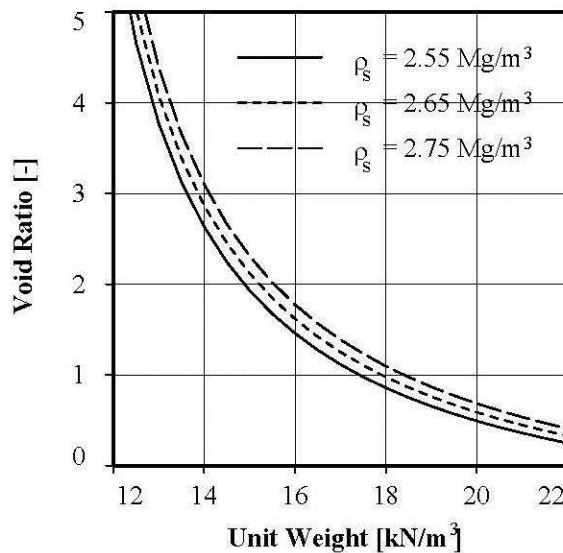
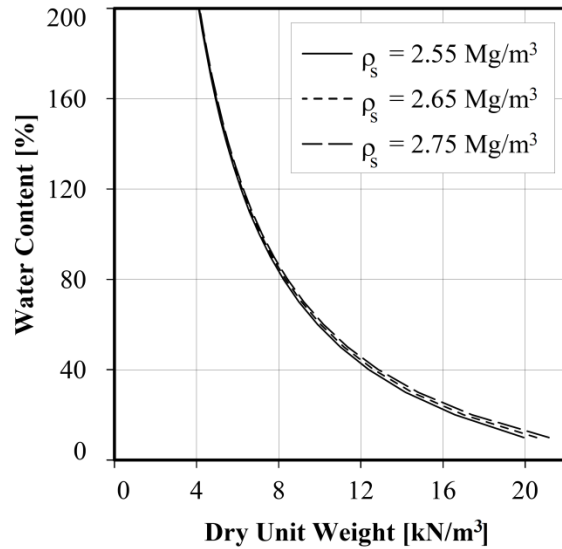
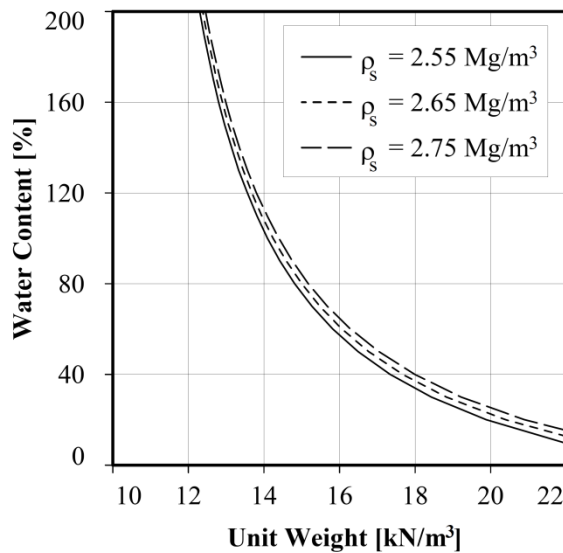
Test references: BS 1377-2:1990, ASTM D7263-09(2018)e2, ISO 17892-2:2014, ISO 19901-8:2014

Unit Weight derived from Water Content

Water content (w) measurement allows estimation of soil unit weight (γ) and unit weight of dry soil (γ_d) on fully saturated samples. This practice requires input on density of solid particles (ρ_s) and presumes saturation of 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 diagrams below presume saturation of non-saline pore water.

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: BS 1377-2:1990, ASTM D854-14, ISO 17892-3:2015, ISO 19901-8:2014

Grain Shape

Grain shape is determined by microscopic comparison of both grain roundness and sphericity with standard grain shapes. The standard shapes are presented together with the test results.

Test references: ISO 19901-8:2014, Krumbein and Sloss (1963)

Particle Size Analysis

Particle size analysis can be performed by means of sieving and/or hydrometer readings. 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 hydrometer readings 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.

The hydrometer method allows measurement of the density of a suspension consisting of fine-grained 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 ($25^\circ \pm 0.5^\circ$). 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 is measuring the density of the suspension. These calculations require a value for the density of solid particles. Generally, a value of 2.65 t/m^3 is assumed. When other values are used, this is included in the

laboratory report. The hydrometer 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 hydrometer readings and sieving normally requires data harmonising in the area of overlap, i.e. near the 0.06 mm particle size.

Test references: BS 1377-2:1990, ASTM D6913/D6913M-17, ASTM D7928-17, ISO 17892-4:2016, ISO 19901-8:2014

Percentage Fines

The Percentage Fines test identifies the proportions of fine grained (< 0.06 mm for BS/ISO and < 0.075 mm for ASTM) and coarse-grained (> 0.06 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 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: BS 1377-2:1990, 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-grained soils. They are expressed as water content, in percent. 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 part of soil is 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 one-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 and ASTM standards define plasticity classifications (e.g. low, high). ISO 14688-2:2017 gives a (non-numerical) plasticity classification. Fugro results can include a numerical interpretation of the ISO plasticity classification.

Test references: BS 1377-2:1990, BS 5930:2015, ASTM D2487-17, ASTM D4318-17e1, ISO 19901-8:2014, ISO 14688-2:2017, ISO 17892-12:2018

Minimum Index Unit Weight

The minimum index unit weight (γ_{dmin}) 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 of either 70 ml or 550 ml volume.

Test reference: BS 1377-4:1990, ASTM D4254-16

Maximum Index Unit Weight – Impact Compaction

The maximum index unit weight (γ_{dmax}) of cohesionless soil is determined from the mass of oven-dry, compacted soil in a standard mould. The soil is compacted in 5 layers, with each layer being subjected to respectively 5, 10, 20, 40 and 80 blows from a standard, hand-held hammer.

Equipment dimensions are as follows. Preference is given to the large mould, but application depends on size of sample.

Equipment	70.5 ml mould	554 ml mould
Hammer mass [g]	185	750
Drop height [mm]	300	390
Cross-sectional area [mm ²]	1006	38,500

Reference: DGI Product Sheet #000 96-07-02

Maximum Index Unit Weight – Vibrating Hammer

The maximum index unit weight (γ'_{dmax}) 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 Method

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 measurements using this procedure.

Test references: 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 D2974-14, NEN 5754, ISO 19901-8:2014

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 (CO₂) upon reaction of the soil with hydrochloric acid (HCl). The carbonate content is calculated from calibration values and expressed as a percentage of dry mass of the original soil.

Test reference: ISO 10693:2014, ISO 19901-8:2014

Carbonate Content – Rapid Titration Method

The carbonate content is determined by using a dried or a natural soil specimen and measuring the pressure of dissipated carbon dioxide (CO₂) 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

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 (CO₂) 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 reference: ASTM D4373-14, ISO 19901-8:2014

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 % SO₃ or % SO₄.

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 (m)g/L or in terms of percentage of water-soluble sulphate in the soil sample.

If water-soluble sulphate content is provided in (m)g/L SO₃, one can convert this to (m)g/L SO₄ by multiplying SO₃ 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 SO₃ (or SO₄) (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 references: 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 Conductivity 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 sample, 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 by probe measurements, 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.

Thermal conductivity of soil is calculated from a suitable part of a temperature versus time curve (typically the last 2/3 of the measurements), 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 D5334-14

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:

$$\rho = \frac{RA}{1000L}$$

where:

ρ	=	Soil electrical resistivity [$\Omega \cdot m$]
R	=	Measured resistance [$\Omega \cdot s$]
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 gas 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.

References: BS 1377-6:1990, 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.

References: BS 1377-5:1990, ASTM D2434-19, ISO 17892-11:2019, ISO 19901-8: 2014

Compressibility 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, BS 1377-5:1990, 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 measurements 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 , and the coefficient of vertical permeability k_v .

Test reference: ASTM D4186/D4186M-12, ISO 19901-8:2014

Strength Index Testing

Torvane and Pocket Penetrometer

The torvane and pocket penetrometer are small hand-held instruments for rapid strength index testing of fine grained (cohesive) soils. 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 readings of up to 900 kPa can be taken.

Test reference: ASTM D8121/D8121M-19, ISO 22475-1:2006, 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 reference: ISRM (2014); Ulusay & Erguler (2012)

Fall Cone

The fall cone is a rapid index test for determining undrained shear strength of undisturbed or remoulded specimens of cohesive soil. 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 sample 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.

Reference: ISO/TS 17892-6, 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 measurements of undrained shear strength are possible:

- Intact: undisturbed undrained shear strength as measured on an intact specimen.
- Intact-residual: measured post-peak during initial shearing of an intact specimen.
- Intact-vane-remoulded: measured after multiple rotations of the hand vane after completion of the intact test.
- Hand-remoulded: steady state (post-peak if exists) resistance of a hand-remoulded test specimen.
- Hand-remoulded-cane-remoulded: steady state resistance of a hand-remoulded specimen measured after applying multiple vane rotations.

Different values of the remoulded shear strength are often obtained from the different measurement methods.

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 measurements of undrained shear strength are possible:

- Intact: undisturbed undrained shear strength as measured on an intact specimen.
- Intact-residual: measured post-peak during initial shearing of an intact specimen.
- Intact-vane-remoulded: measured after multiple rotations of the vane after completion of the intact test.
- Hand-remoulded: steady state (post-peak if exists) resistance of a hand-remoulded test specimen.
- Hand-remoulded-vane-remoulded: steady state resistance of a hand-remoulded specimen measured after applying multiple vane rotations.

Different values of the remoulded shear strength are often obtained from the different measurement methods.

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 (BS 1377) or at 60°/min to 90°/min (ASTM D4648) 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: BS 1377-7:1990, ASTM D4648/D4648M-16, ISO 19901-8:2014, ISO 17892-8:2018

Unconsolidated Undrained Triaxial (UU)

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 1000 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 undrained shear strength 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.

To determine strength sensitivity, the test may be repeated on remoulded (compacted) specimens. 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. The sensitivity is the ratio of shear strength of undisturbed soil to shear strength of remoulded soil, s_u/s_{ur} .

Test references: ASTM D2850-15, ISO 17892-8:2018, ISO 19901-8:2014

Strength and Stiffness Testing

Ring Shear - Soil/Steel Interface

Ring shear interface tests are performed on remoulded or reconstituted (compacted) soils to infer the residual friction angle, also called the constant volume friction angle (δ_{cv}), on a 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 rotational shear.

First, the sample is consolidated to selected stress conditions. Then, it is sheared at a rate of 500 mm/min (fast shear), followed by 50 mm/min, up to a relative displacement of at least one metre. The sample is then resting for a period of 24h and after that is again consolidated to its selected stress conditions. Finally, the sample is sheared at a slower rate of 0.018 mm/min under drained conditions.

The presentation of the test results includes a plot of stress ratio and angle of shearing resistance versus displacement, both for fast and slow shear.

Test reference: BS 1377-7:1990, Jardine et al. (2005) (Appendix A), ISO 19901-8:2014, ISO 17892-10:2018

Direct Simple Shear (DSS)

Simple shear 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 – Soil/Soil Interface

Direct shear testing (or shear box testing) is a method for determining drained soil resistance (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 reference: BS 1377-7:1990, ISO 19901-8:2014, ISO 17892-10:2018

Consolidated Undrained Triaxial (CIU and CAU)

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 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 $\sigma_1 - \sigma_3$;
- BSI (1990) 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
2. maximum stress ratio q/p' .

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 $A = 0$.

A secant angle of internal friction, ϕ' , is determined from $q = Mp'$ where $M = (6\sin\phi')/(3-\sin\phi')$ or $\sin\phi' = 3M/(6+M)$ 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, $\sin\phi' = t/s'$. 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, BS 1377-8: 1990, ISO 19901-8:2014, ISO 17892-9:2018

Consolidated Drained Triaxial (CID and CAD)

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 $\sigma_1 - \sigma_3$;
- BSI (1990) s' - t space, with s' defined as $(\sigma'_1 + \sigma'_3)/2$ and t as $(\sigma_1 - \sigma_3)/2$.

A secant angle of internal friction, ϕ' , is determined from $q = Mp'$ where $M = (6\sin\phi')/(3-\sin\phi')$ or $\sin\phi' = 3M/(6+M)$ 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, $\sin\phi' = t/s'$. 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 reference: ASTM D7181-20; BS 1377-8: 1990, 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 the 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. Penetration

resistance is measured just above the T-bar or the ball. Some systems measure penetration resistance 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 $300 \text{ mm} < h < 600 \text{ mm}$ and a diameter of typically $> 300 \text{ mm}$. 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.

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.

$$q_T \text{ or } q_B = Q_p/A_1 - (A_2/A_1)u_0 - (A_3/A_1) \cdot (\sigma_{v0} - u_0)$$

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 = axially measured 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_{v0} = 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, measured 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 10th cycle. Values for $N_{T,r}$ (and $N_{B,r}$) are in the order of 13. Determination of derived values for $s_{u,r}$ is optional.

Test reference: In-house, ISO 19901-8:2014

Piezoceramic Bender Elements

Bender elements are small piezo-electric elements 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 and density of the soil.

Test reference: ASTM D8295-19, ISO 19901-8:2014

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Cyclic and Dynamic Laboratory Tests

Testing Practice

General

This document summarises geotechnical laboratory test methods for cyclic and dynamic soil behaviour.

Fugro strives to arrange testing in registered laboratories with formal accreditation.

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 (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.

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.

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. Usually, a reasonable estimate of test feasibility is possible at the time of sampling. A further refinement is possible in the laboratory prior to testing and, in some cases, only after testing. The limitations of feasibility estimates may lead to rejection of samples for testing upon inspection in the laboratory or may result in appropriate comments on test results after completion of testing.

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.

Sample Quality

The adopted classification system for sample quality is according to BSI (2015) and ISO (2006, 2014). The classification system recognises 5 classes on the basis of feasibility of specific geotechnical identification and laboratory tests. A summary of these classes is as follows:

- Class 1: undisturbed: strength, stiffness and consolidation
 - Class 2: undisturbed: permeability, unit weight, boundaries of strata - fine
 - Class 3: disturbed: water content
 - Class 4: disturbed: particle size analysis, Atterberg limits, boundaries of strata - broad
 - Class 5: disturbed: sequence of layers
- The higher class includes the laboratory tests of the lower class.

An indication of intact (undisturbed) sample quality may be obtained from re-compression of a test specimen, for example in oedometer, direct simple shear or triaxial apparatus. Table 1 presents a method recommended by ISO (2014) based on $\Delta e/e_0$.

Here, Δe represents the change in void ratio Δe from an initial laboratory value (e_0) 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: 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

The presented criteria are based on tests on marine clays 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 2 presents an alternative indication of intact (undisturbed) sample quality according to Terzaghi et al. (1996). Here, volumetric strain is derived from an initial laboratory specimen volume and the specimen volume upon re-compression to estimated in situ stress conditions. 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	

The $\Delta e/e_0$ and ε_v 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 $\Delta e/e_0$ and ε_v .

The $\Delta e/e_0$ and ε_v 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 $\Delta e/e_0$ and ε_v should exclude secondary consolidation. In practice, no correction for secondary consolidation will be applied. This practice underestimates undisturbed sample quality, particularly for incremental loading oedometer tests with 24 hour load increments and longer.

Strength Testing – Cyclic

Cyclic Direct Simple Shear

A cyclic direct simple shear (CSS) test is carried out on a cylindrical specimen of 50 mm or 67 mm diameter and 19 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.

Most types of test apparatus have no provision for controlling saturation or measuring pore pressure during the test; back pressure cannot be applied to the specimen.

Cyclic and monotonic (static) shearing may be performed under either constant volume or constant vertical stress conditions. The constant volume condition is equivalent to the undrained condition for a saturated specimen. The constant volume condition is achieved by varying the normal load applied to the specimen or by fixing the vertical loading ram in place. It is generally accepted that the change in vertical stress during shearing can be equated to the change in pore water pressure that would have occurred in a truly undrained test. The constant vertical stress condition is equivalent to the drained condition for a saturated specimen. The constant vertical stress condition is achieved by continuous adjustment of the specimen height during shearing, so that the vertical stress acting on the specimen remains constant.

For cohesionless specimens, cyclic pre-shearing may optionally be applied to simulate densification of the soil under low amplitude wave loading. If required, on completion of the consolidation stage, a nominally low amplitude (assigned based on foundation considerations and specimen depth) cyclic shear stress (τ_{cy}) is applied to the specimen under constant vertical stress conditions.

The CSS test may be performed as a stress-controlled or strain-controlled test. On completion of the consolidation stage, or following pre-shearing (if required), a combination of either an initial average shear stress (τ_{av}) and τ_{cy} is applied to the specimen in stress-controlled tests, or an initial average shear strain (γ_{av}) and cyclic shear strain (γ_{cy}) in strain-controlled tests. The cyclic shear stress, τ_{cy} , is defined as one half of the maximum change in shear stress over one load cycle. Where required, τ_{av} is applied to the specimen prior to the application of cyclic shear stress or cyclic shear strain. This initial shear stress may be applied under constant volume or constant vertical stress conditions. After application of the initial stress, the specimen is left to equalise for a period of time (nominally 1 hour). A sinusoidal τ_{cy} of specified amplitude around the initial τ_{av} value (stress-controlled tests) or sinusoidal cyclic shear strain (γ_{cy}) of specified amplitude (strain-controlled tests) is then applied to the specimen. The cyclic loading is applied at a constant frequency which is typically 0.1 Hz or 1 Hz.

Common practice for stress-controlled CSS tests covers:

- Constant cyclic stress ratio τ_{cy}/σ'_{ref} , where σ'_{ref} is the reference stress for which commonly the peak shear stress from a static direct simple shear test is chosen;
- Constant average stress ratio (τ_{av}/σ'_{ref}) per test;
- A series of tests at various stress ratios.

An optional procedure is to vary the stress level and to apply parcels of load cycles. Drainage/consolidation between parcels can be allowed to simulate specific geotechnical design conditions.

Cyclic loading is typically terminated when one of several agreed criteria are reached. Usually a maximum number of cycles or a

limit average or cyclic shear strain (e.g. 15 %) is used for this purpose.

Following termination of the cyclic shearing stage, the specimen can (optionally) be sheared monotonically to failure to determine post-cyclic shear behaviour. A shear stress is applied to the specimen at a constant shear strain rate via the base loading platen. Monotonic shearing may be performed under constant volume or constant vertical stress conditions. No volume change is permitted between cyclic shearing and post-cyclic monotonic shearing, unless indicated otherwise.

Test results typically consist of the following diagrams:

- Number of loading cycles versus (a) shear stress, (b) shear strain, (c) change in vertical stress (constant volume test), (d) normalised change in vertical stress (constant volume test), and (e) vertical strain (constant vertical stress test);
- Vertical stress versus shear stress;
- Shear strain versus shear stress.

Various data processing options are available to suit specific project requirements.

Test reference: ASTM D8296-19, ISO 19901-8:2014

Cyclic Consolidated Undrained Triaxial

Consolidated cyclic triaxial tests with pore water pressure measurement are typically performed on intact or reconstituted soil specimens with a height to diameter ratio of approximately 2:1. Where possible, specimens are tested at full size (diameter of original sample). Cyclic loading is applied under undrained conditions.

The specimen set-up and testing procedures are essentially similar to the static triaxial test, except that the axial or radial stress is applied cyclically, generally at a frequency of 0.1 Hz or 1 Hz.

Where cyclic pre-shearing of a reconstituted coarse-grained specimen is required to model densification under small magnitude cyclic loading, this is applied to the specimen after completion of the consolidation stage. The drainage lines are left open and a small cyclic deviator stress (q_{cy}) is applied to the specimen for up to 200 loading cycles.

The cyclic triaxial test may be performed as a stress-controlled or strain-controlled test. On completion of the consolidation stage, or following pre-shearing (if required), the drainage lines are closed and a combination of either an initial (average) deviator stress (q_{av}) and q_{cy} (for stress-controlled tests) is applied to the specimen. The cyclic deviator stress, q_{cy} , is defined as one half of the maximum change in deviator stress over one load cycle. Where required, q_{av} is applied to the specimen prior to the application of cyclic deviator stress or cyclic axial strain. This q_{av} is applied undrained and left to equalise for a period of time (nominally 1 hour). A sinusoidal q_{cy} of specified amplitude around the initial q_{av} value (stress-controlled tests) or sinusoidal cyclic axial strain (ε_{cy}) of specified amplitude (strain-controlled tests) is then applied to the specimen.

Common practice for stress-controlled cyclic triaxial tests covers:

- Constant cyclic deviator stress ratio q_{cy}/σ'_{ref} , where σ'_{ref} is the reference stress for which commonly the applied effective vertical consolidation stress is chosen;
- Constant cyclic average deviator stress ratio (q_{av}/σ'_{ref}) per test;
- A series of tests at various stress ratios.

An optional procedure is to vary the stress level and to apply parcels of load cycles. Drainage/consolidation between parcels can be allowed to simulate specific geotechnical design conditions.

Cyclic loading is typically terminated when one of several agreed criteria are reached. Usually a maximum number of cycles or a limit average axial strain (ϵ_{av}) or cyclic axial strain (ϵ_{cy}), for example 15 %, is used for this purpose.

Following termination of the cyclic loading stage the specimen may be sheared statically to determine post-cyclic undrained shear strength. The specimen is sheared at an appropriate strain rate, in the range 0.001 %/hour to 10 %/hour, such as to allow an equalised pore water pressure response throughout the specimen. Drainage is not permitted between cyclic loading and post-cyclic monotonic shearing.

Test results typically consist of the following diagrams:

- Number of loading cycles versus (a) deviator stress, (b) pore pressure ratio, (c) excess pore pressure, and (d) axial strain;
- Axial strain versus deviator stress;
- p' - q and s' - t stress paths.

Various data processing options are available to suit specific project requirements.

Test references: ASTM D3999M-11e1, ASTM D5311M-13, ISO 19901-8:2014

Stiffness Testing – Dynamic

Resonant Column

The resonant column test allows determination of shear modulus and damping properties at small strain of a cylindrical soil specimen by applying torsion. The torsion apparatus and specimen are enclosed in a triaxial cell so that controlled states of stress and pore water pressure can be applied. The test may be carried out on intact or reconstituted specimens with a height to diameter ratio of typically 2:1.

Test procedures include specimen saturation, consolidation, and vibration excitation.

Saturation is obtained using standard triaxial test techniques. The degree of saturation is checked by the pore pressure response to controlled variations in cell pressure (B -value method). For a reconstituted cohesionless specimen, the saturation phase may be preceded by flushing the specimen with CO₂ gas.

In case of isotropic consolidation, the specimen is usually consolidated to a stress level equivalent to the mean stress estimated for the appropriate sample depth. For anisotropic consolidation, the sample is consolidated to the estimated vertical and horizontal effective stresses. A series of consolidation stages may be adopted to simulate a stress history.

During torsional excitation, the specimen is fully fixed at the bottom end and an electromagnetic oscillator applies a sinusoidal torsional motion or longitudinal vibration to the top. A range of torques is applied resulting in average shear strain amplitudes typically in the range 0.0001 % to 0.1 %. The initial torque is re-applied after each stage to check for any degradation of the specimen.

Resonance stages may be applied under undrained conditions. The resonant frequency is obtained by increasing the excitation frequency from a minimum of about 10 Hz until the sinusoidal excitation moment is in phase with the velocity of the top platen. The lowest frequency at which this occurs is defined as the resonant frequency. The shear strain amplitude and shear

modulus of the specimen are calculated from the peak acceleration and frequency of the top cap at resonance. The damping ratio is obtained by means of the free vibration decay method.

Reference: ASTM D4015-15e1, ISO 19901-8:2014

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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 (2012), ISO (2012) and ISO (2014). BS 5930 (BSI, 2015) refer to ISSMGE (1999). 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;
- 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 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: ring or special projections fixed on the outside of the push rods, with an outside diameter larger than the base of the cone, to reduce soil friction acting on the push rods.

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.

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

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

Cone Penetrometer

Typical features of Fugro penetrometers (Figure 1) include:

- Cone base areas of 500 mm², 1000 mm² or 1500 mm²; other sizes are also in use, e.g. 3300 mm²;
- 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) or at the cylindrical extension of the cone (u_2). Multiple-sensor penetrometers (u_1 , u_2 and u_3) are also available. The u_3 location is immediately above the friction sleeve;
- Incliner;
- Storage of signals from the penetrometer in digital form for subsequent computer-based processing and presentation.

Procedure

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

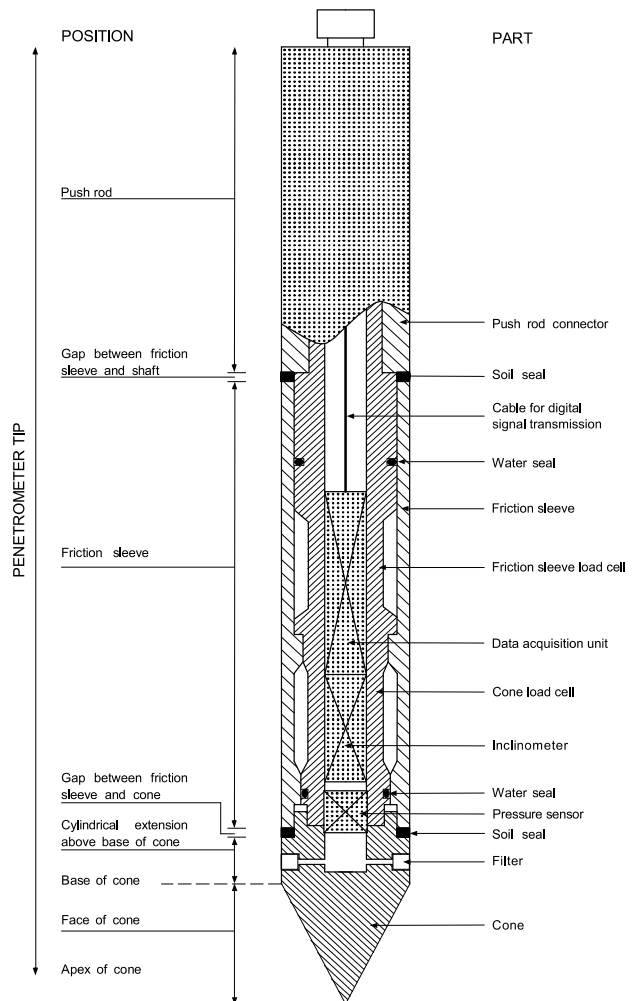


Figure 1: Piezocone Penetrometer

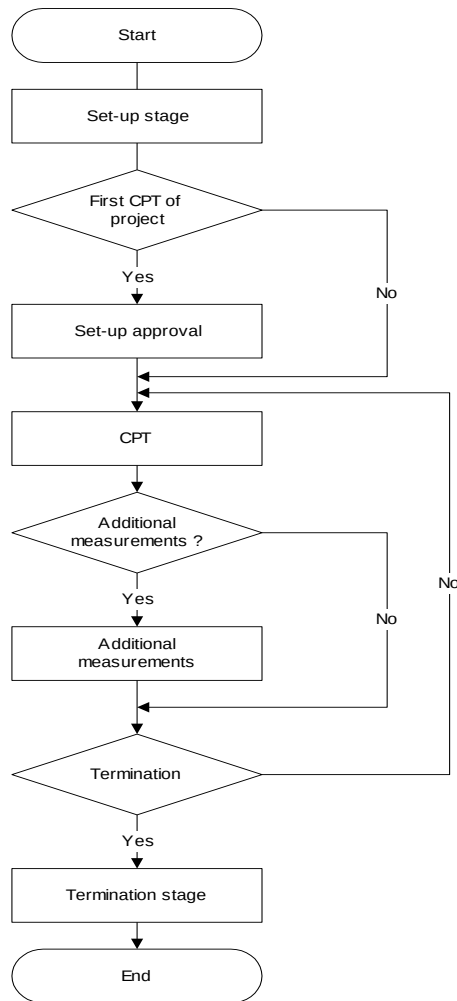


Figure 2: Flow chart

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 5o 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 accuracy class or application class, type of cone penetrometer and data processing/ submission. Table 1 presents ISSMGE accuracy classes and Tables 2 and 3 summarise ISO application classes. 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. The concept of application classes considers intended soil conditions for

selection of an application class. For example, Application Class 1 of ISO (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 ground surface or seafloor as reference. They are uncoupled from uncertainty of spatial position below ground surface or seafloor.

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.

Table 1: Accuracy Classes (ISSMGE, 1999)

Accuracy Class	Parameter	Allowable Minimum Accuracy	Maximum Length between Measurements
1	Cone resistance Sleeve friction Pore pressure, Inclination Penetration depth	50 kPa or 3 % 10 kPa or 10 % 5 kPa or 2 % 2° 0.1 m or 1 %	20 mm
2	Cone resistance Sleeve friction Pore pressure Inclination Penetration depth	200 kPa or 3 % 25 kPa or 15 % 25 kPa or 3 % 2° 0.2 m or 2 %	20 mm
3	Cone resistance Sleeve friction Pore pressure Inclination Penetration depth	400 kPa or 5 % 50 kPa or 15 % 50 kPa or 5 % 5° 0.2 m or 2 %	50 mm
4	Cone resistance Sleeve friction Penetration length	500 kPa or 5 % 50 kPa or 20 % 0.1 m or 1 %	50 mm

Table 2: Application Classes (ISO, 2012)

Application Class	Parameter	Allowable Minimum Accuracy	Maximum Length between Measurements
1	Cone resistance Sleeve friction Pore pressure, Inclination Penetration length	35 kPa or 5 % 5 kPa or 10 % 10 kPa or 2 % 2° 0.1 m or 1 %	20 mm
2	Cone resistance Sleeve friction Pore pressure, Inclination Penetration length	100 kPa or 5 % 15 kPa or 15 % 10 kPa or 3 % 2° 0.1 m or 1 %	20 mm
3	Cone resistance Sleeve friction Pore pressure, Inclination Penetration length	200 kPa or 5 % 25 kPa or 15 % 50 kPa or 5 % 5° 0.2 m or 2 %	50 mm
4	Cone resistance Sleeve friction Penetration length	500 kPa or 5 % 50 kPa or 20 % 0.2 m or 2 %	50 mm

Table 3: Application Classes (ISO, 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 %

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;
- 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.

Special apparatus and procedures may apply to:

- Specific additional measurements (for example shear wave velocity);
- Specific applications (for example offshore tests and measurements for application/accuracy Classes 1 and 2).

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_n , q_n , B_q , Q_n , Q_{en} , F_r and I_c for tests with pore pressure measurements;
- Optionally, inclination i for tests with inclination measurements;
- Standard graphical format and optional ASCII and AGS formats.

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_i 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.

Historically, the bottom of the borehole was used as the reference level of downhole 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:

$$z = \int_0^l \cos i \cdot dl$$

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

$$f_t = f_s - \frac{(u_2 \cdot A_{sb} - u_3 \cdot A_{st})}{A_s}$$

or simplified to:

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

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

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_t/q_t). The ratio f_t/q_t applies if f_t is known.

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

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

where:

P_a = atmospheric pressure

- 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.
- 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:

Ground surface/seafloor: Downhole (historic):

$$q_t = q_c + (1-a)u_2 \text{ or } q_t = q_c + (1-a)u_2 + u_{oi} \text{ or } q_t = q_c + (1-a)\{K(u_1 - u_o) + u_o\} \text{ or } q_t = q_c + (1-a)\{K(u_1 + u_{oi} - u_o) + u_o\} + au_{oi}$$

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

u_{oi} = 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.

Ground surface/seafloor: Downhole (historic):

$$K = (u_2 - u_o) / (u_1 - u_o) \quad K = (u_2 + u_{oi} - u_o) / (u_1 + u_{oi} - u_o)$$

The term $u_2 - u_o$ or $u_2 + u_{oi} - u_o$ refers to excess pore pressure (with respect to hydrostatic pore pressure). Common symbols for excess pore pressure are du_2 or Δu_2 . Similarly, du_1 or Δu_1 may represent the term $u_1 - u_o$ or $u_1 + u_{oi} - u_o$.

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 can be estimated from:

$$K = 0.91e^{-0.09Q_t^{0.47}} \left(\frac{1}{1 + F_r(0.17 + 0.061(Q_t - 21.6)^{1/3})} - e^{-2F_r} \right) \quad (\text{Peuchen et al., 2010})$$

- 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.

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

where:

σ'_{vo} = 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:

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

$$n = 0.381 (I_c) + 0.05 (\sigma'_{vo} / P_a) - 0.15 \text{ and } n \leq 1 \quad (\text{Zhang et al., 2002})$$

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

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

- F_r = f_t / q_n = normalized friction ratio.

- B_q = pore pressure ratio.

Ground surface/seafloor: Downhole (historic):

$$B_q = K(u_1 - u_o) / q_n \text{ or } B_q = K(u_1 + u_{oi} - u_o) / q_n \text{ or } B_q = (u_2 - u_o) / q_n \text{ or } B_q = (u_2 + u_{oi} - u_o) / q_n$$

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.

Accuracy Classes and Application Classes

Cone penetration test standards can follow a "prescriptive" approach, whereby specific detailed measures provided a "deemed to comply" practice. ASTM (2012) is an example of this approach. ISSMGE (1999) and ISO (2012, 2014) specify "performance" criteria for cone penetration test measurements. The ISO standard on metrological confirmation (ISO, 2003) provides the general framework for assessment of performance compliance.

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.
- Accuracy Class 3 and 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.

So-called "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. The zero drift of the measured parameters 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 (Tables 1, 2 or 3).

Differences in interpretation about compliance with the ISSMGE and ISO box values for accuracy became apparent after publication of ISO 22476-1:2012 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 (Peuchen and Parasie, 2019).

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.

Accuracy considerations for strongly layered soils should allow for heat flux phenomena. Heat flux gives an apparent shift in cone resistance (Post and Nebbeling, 1995). For example, friction in dense sand causes a cone to heat by about 1°C/MPa cone resistance. Resulting heat flux decreases 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 decrease lasting about 5 minutes. Ambient temperature compensation systems cannot avoid heat flux effects. Penetration interruption can serve as mitigation measure.

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 (Lunne et al., 1997; Peuchen and Terwindt, 2014). 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;
- 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 parameters 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.

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

Offshore definition of the seafloor (ground surface) is difficult for extremely soft ground at seafloor (Peuchen et al., 2005). 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 according to ISO (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 tests.

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 , and thermal conductivity, k	K, W/(m·K)
Seismic Cone Penetrometer (SCPT)	S-wave velocity, v_s , and P-wave velocity, v_p	m/s
Cone Pressuremeter (CPMT)	Shear 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 and vertical angle ϕ	μT , °, °
Hydraulic Profiling Tool (HPT)	Permeability, k	m/s
S = Siemens	s = second	
m = metre	Pa = Pascal	
K = Kelvin (or °C)	CPS = counts per second	
W = Watt	T = Tesla	

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Cone Penetration Test Interpretation

Scope

This document presents a summary of interpretation methods for cone penetration test (CPT) results. The project-specific selection of methods depends on the agreed project requirements. Some of the methods suit computer-based interpretation of CPT data records.

Parameter Interpretation

Interpretation of cone penetration test results helps provide parameters for geotechnical 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.

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.

The interpretation techniques discussed below are subject to limitations such as:

- CPT measurements, including measurement uncertainty (Peuchen and Terwindt, 2014 & 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.
- Most interpretation methods apply a transformation model to "conventional" sands (drained soil behaviour) and clays (undrained soil behaviour). Conventional methods 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 non-conventional soils warrant a more specific approach.
- Drained or undrained behaviour for the geotechnical analysis at hand may or may not coincide with respectively drained or undrained behaviour during fixed-rate penetration testing. This interpretation difficulty remains largely unresolved at this time.
- 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.

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 total 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 and 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).

Soil Behaviour Identification

Identification of soil stratigraphy in terms of general soil behaviour (and to a lesser degree soil type) is a more important feature of CPT than other investigation techniques.

Figures 1 and 2 show soil behaviour identification according to procedures given by Robertson (2009), representing an update of Robertson (1990) 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 possible for $1 \leq Q_{tn} \leq 1000$, $0.1 \leq F_r \leq 10$ and $-0.2 \leq B_q \leq 1.4$. The procedures require piezocone test data:

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

$$F_r \text{ or } nR_f = \frac{f_s}{q_t - \sigma_{vo}} 100\% \quad B_q = \frac{u - u_0}{q_t - \sigma_{vo}}$$

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):

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

Robertson and Wride (1998) defined soil behaviour type index I_c (Figure 2) 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 and Wride, 1998). Cone penetration in soils with $2.05 < I_c < 2.60$ is often partially drained.

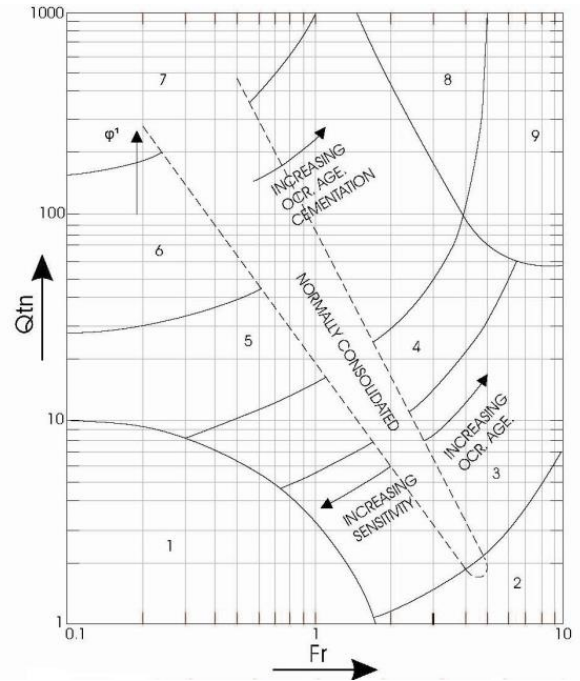


Figure 1: Classification chart Robertson (2009)

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

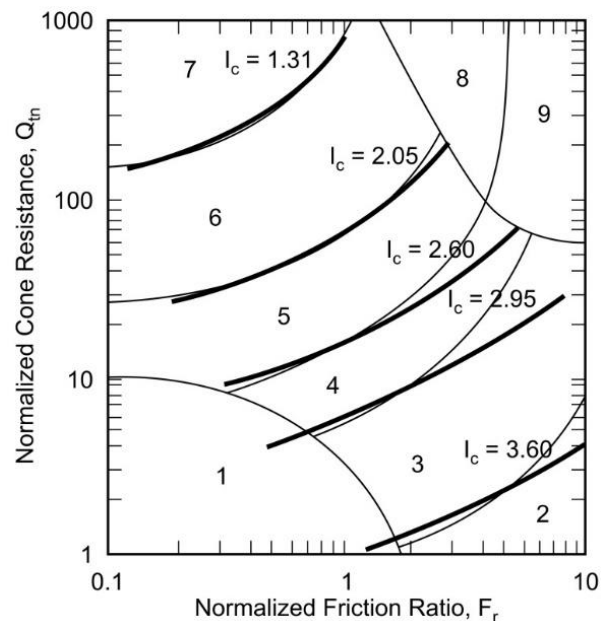


Figure 2: Soil behaviour type index I_c superimposed on Robertson (2009) classification chart

Figure 3 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.

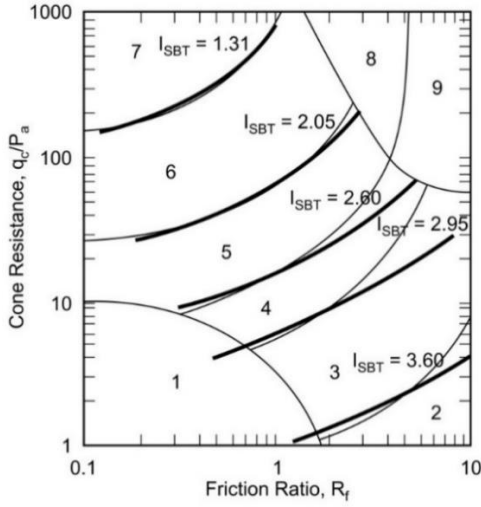


Figure 3: Robertson (2010) classification chart including I_{SBT}

Figure 4 presents a classification chart focusing on contractive and dilative soil behaviour, according to Robertson (2016a). The equations for the contractive-dilative boundary (CD) and modified soil behaviour type index (I_B) are as follows:

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

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

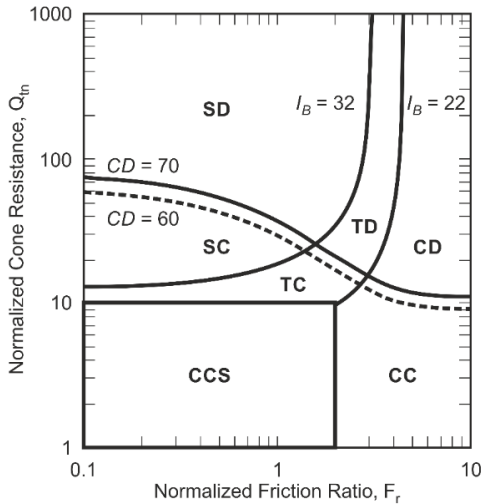


Figure 4: Classification chart according to Robertson (2016a)

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

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.

Sand Model

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: $\gamma_{ref} = 19 \text{ kN/m}^3$, $\beta = 4.12$, $q_{t,ref} = 5 \text{ MPa}$, 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 6). The correlation considers a normalized cone resistance ($q_{c1N,hm}$) and a soil behaviour type index ($I_{c,hm}$) as follows:

$$v_s = 0.0831 q_{c1N,hm} (\sigma'_{vo}/P_a)^{0.25} e^{(1.786 I_{c,hm})} \quad (\text{Hegazy and Mayne, 2006})$$

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 .

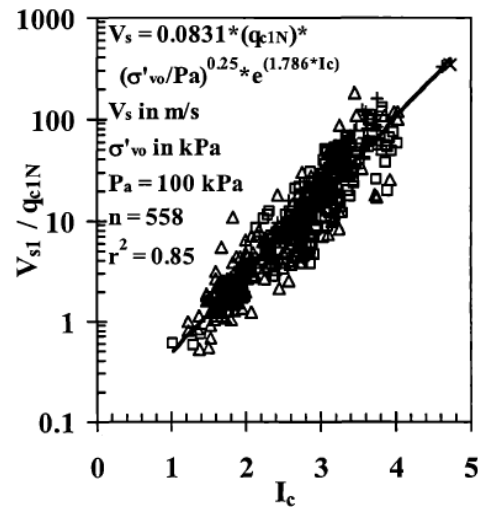


Figure 6: $v_s - q_c$ correlation according to Hegazy and Mayne (2006)

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):

$$v_s = [\alpha_{vs}(q_t - \sigma_{vo})/P_a]^{0.5} \text{ where } \alpha_{vs} = 10^{(0.55 \cdot I_c + 1.68)} \quad (\text{Robertson and Cabal, 2015})$$

where shear wave velocity v_s is in m/s and total 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 (SCPT) performed in quaternary deposits of the predominantly silica Po river sand and Gioia Tauro sand with gravel.

$$v_s = 277 q_c^{0.13} \sigma'_{vo}{}^{0.27} \quad (\text{Baldi et al., 1989})$$

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):

$$v_{s1} = v_s \cdot (P_a/\sigma'_{vo})^{0.25} \quad (\text{Robertson and Cabal, 2015})$$

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 $\sigma'_{ho} = K_o \sigma'_{vo}$ is generally unknown. Usually, it is necessary to consider a range of conditions for K_o (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_o .

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 empirical approach can be applied for direct estimation of K_o based on Mayne (2017):

$$K_o = 0.4 \sqrt{OCR}$$

$$\text{using: } OCR = \frac{\sigma'_p}{\sigma'_{vo}} \quad \sigma'_p = 0.33 \cdot q_n^{m'} \quad m' = 1 - \frac{0.28}{1 + \left(\frac{I_c}{2.65}\right)^{2.5}}$$

where OCR is overconsolidation ratio, σ'_p is effective preconsolidation stress, σ'_{vo} is effective in situ vertical stress, q_n is net cone resistance in kPa and I_c is soil behaviour type index.

The K_o -OCR relationship represents a simplification of $K_o = (1 - \sin \phi') OCR^{\sin \phi'}$ proposed by Mayne and Kulhawy (1982). Mayne and Kulhawy (1982) investigated mechanical overconsolidation of reconstituted laboratory specimens for over 170 different soils. For sands, it can be shown that the $K_o = 0.4 \sqrt{OCR}$ equation provides similar statistics to the Mayne and Kulhawy correlation.

$$K_o = (1 - \sin \phi') \cdot OCR^{\sin \phi'}$$

Figure 5 presents an approximate CPT-based correlation for K_o according to Robertson (2016b). K_o limits are typically set to 0.5 and 2. Linear interpolation is applied for the region between $K_o = 0.5$ and $K_o = 2$.

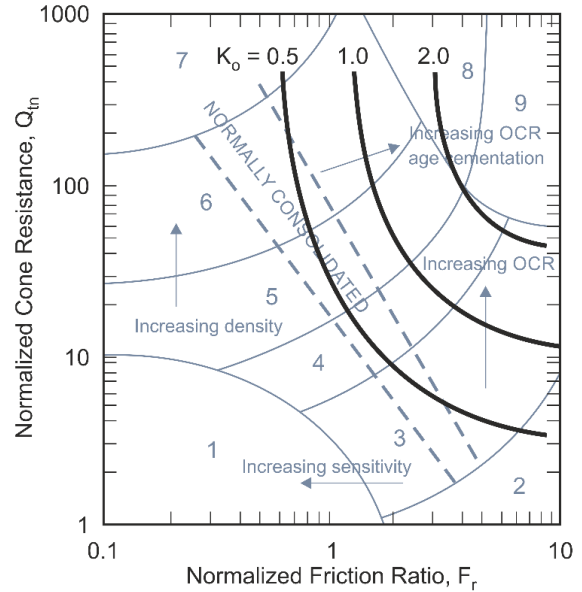


Figure 5: K_o correlation according to Robertson (2016b)

No laboratory study can fully capture in situ behaviour. Particularly, K_o may be underestimated if effects such as ageing and cyclic loading are relevant.

In general, in situ K_o values are limited to the range $K_o = 0.5$ to $K_o = 1.5$. For many situations, K_o values are believed to be relatively low at greater depths (say $K_o < 1$ for depths exceeding 50 m). Jamiolkowski et al. (2003) recommend using a limiting value $K_o = 1$ 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.

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_o values for calibration testing;
- results applicable to reconstituted sand samples, sample preparation method and soil stress history simplifications.

Jamiolkowski et al. (2003) proposed the following relationship between q_c and D_r for normally and overconsolidated silica (dry) sands:

$$D_{r(dry)} = \frac{1}{2.96} \ln \left[\frac{\frac{q_c}{P_a}}{24.94 \left(\frac{\sigma'_{vo} \left(\frac{1+2K_a}{3} \right)}{P_a} \right)^{0.46}} \right]$$

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.

Kulhawy and Mayne (1990) proposed:

$$D_r^2 = Q_{tn*}/Q_f \quad \text{where: } Q_{tn*} = \left(\frac{q_t}{P_a} \right) / \left(\frac{\sigma'_{vo}}{P_a} \right)^{0.5}$$

and q_t is corrected cone resistance, P_a is atmospheric pressure, σ'_{vo} 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 their dataset. Robertson and Cabal (2015) suggested $Q_f = 350$ for clean, uncemented, medium compressible quartz sands of about 1000 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.

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. 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.

Calibration chamber test results apply to a limited range of stress conditions; typically:

50 kPa	<	σ'_{vo}	<	400 kPa
0.4	<	K_o	<	1.5

Sample preparation for laboratory chamber tests is usually by means of dry pluviation. Soil stress history application is by mechanical overconsolidation.

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:

- Estimation of in situ stress conditions σ'_{vo} and σ'_{ho}
- Estimation of relative density D_r
- 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).

$$\phi' = 17.6 + 11.0 \log \left(\left(\frac{q_t}{P_a} \right) / \left(\frac{\sigma'_{vo}}{P_a} \right)^{0.5} \right) \quad (\text{Mayne, 2007})$$

Undrained Shear Strength - Sand

Undrained shear strength of cohesionless soil can be important for assessment of cyclic mobility and liquefaction potential. Geotechnical procedures other than the conventional limit state models are employed.

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 and Mayne, 1990)

$$M = q_c * 10^{1.78 - 0.0122 D_r}$$

(overconsolidated sands, Kulhawy and 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 25 for overconsolidated sands. The correlation has been inferred from laboratory conditions; including CPT tests 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 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 low-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 and Stokoe, 1991)

where G_{max} , q_c and σ'_{vo} are in kPa.

CLAY MODEL

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.

$$v_s = 1.75q_c^{0.627}$$

(Mayne and 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, more particular, K_o as $\sigma'_{ho} = K_o \sigma'_{vo}$.

Direct correlations for interpretation of the coefficient of earth pressure at rest K_o are the same as described for the Sand Model.

For normally consolidated clays and silts, K_{onc} may be correlated with angle of internal friction, in accordance with Jaky (1944), or more simply in accordance with Mayne and Kulhawy (1982). The reference angle of internal friction is that obtained from a straight-line approximation of the Mohr-Coulomb failure envelope determined from Consolidated Undrained (CU) triaxial compression tests on undisturbed specimens.

For overconsolidated clays, K_{ooc} may be correlated with angle of internal friction and overconsolidation ratio, in accordance with Mayne and Kulhawy (1982). The plasticity index together with OCR may also be used for preliminary estimates of K_{ooc} as indicated by Brooker and Ireland (1965).

$$K_o = (1 - \sin \phi') OCR^{\sin \phi'}$$

(Mayne and Kulhawy, 1982)

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.

Various analytical and semi-empirical models for interpretation of effective preconsolidation stress from piezocone test data are available. Sandven (1990) presents a summary. The procedures are mostly "experimental" and yet uncommon in practice. Chen and Mayne (1996) presented a direct correlation between net cone resistance and overconsolidation ratio for 205 clay sites around the world, as follows:

$$OCR = 0.317Q_c$$

(Chen and Mayne, 1996)

The overconsolidation ratio may also be inferred 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 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 $s_u = q_c / N_c$.

For specific design situations, a different s_u reference strength should be used. For example, offshore axial pile capacity predictions in accordance with API (2011) recommend s_u to be based on undrained triaxial compression tests, which are likely to yield lower s_u values than in situ vane tests. A site-specific or regional approach should generally be preferred.

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 (2018) 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 407 paired CPT and laboratory test results, 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 = $\phi'/3$.
- Lateral earth pressure coefficient during penetration is equal to the Rankine coefficient of lateral earth pressure under passive conditions.

$$\frac{f_s}{\sigma'_{vo}} = \tan^2\left(45^\circ + \frac{\phi'}{2}\right) \tan\left(\frac{\phi'}{3}\right)$$

(Masood and Mitchell, 1993)

Mayne (2001) proposed an approximation of the Masood and Mitchell equation, as follows:

$$\phi' = 30.8 \left[\log\left(\frac{f_s}{\sigma'_{vo}}\right) + 1.26 \right] \quad (\text{Mayne, 2001})$$

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):

$$\phi' = 29.5 B_q^{0.121} (0.256 + 0.336 B_q + \log N_m) \quad (\text{Mayne, 2001})$$

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.

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.

$$M = 8.25 q_n \quad (\text{Kulhawy and Mayne, 1990})$$

Young's Modulus – Clay

Young's modulus E_u can be derived as follows:

- a. Estimation of undrained shear strength s_u from CPT data, as outlined above.

- b. 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 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.

$$G_{max} = 2.78 q_c^{1.335} \quad (\text{Mayne and Rix, 1993})$$

where G_{max} and q_c are in kPa.

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Marine Reflection Seismics

Introduction

Marine geophysical investigation by seismic reflection methods provides a continuous acoustic record of the seafloor and seabed (ground below seafloor). The acquired data are typically interpreted for site characterisation, particularly mapping of stratigraphic, geomorphological, structural and geohazard features of the seabed.

This document covers 2D high resolution (HR), ultra-high resolution (UHR) and ultra-ultra-high resolution (UUHR) seismic and sub-bottom profiler (SBP) seismic reflection methods.

- HR seismic: method that acquires data containing frequencies between 75 Hz and 300 Hz; expected vertical resolution range: 1 m to 7 m;
- UHR seismic: method that acquires data containing frequencies between 250 Hz and 800 Hz; expected vertical resolution range: 0.5 m to 2 m;
- UUHR seismic: method that acquires data containing frequencies between 750 Hz and 2000 Hz; expected vertical resolution range: 0.2 m to 1 m;
- SBP: method that acquires data containing frequencies between 1000 Hz and 15000 Hz; expected vertical resolution range: < 0.5 m.

These methods are commonly required to provide input for design, installation, operation and decommission of offshore structures (ISO, 2015; ISO, 2019)

Apparatus and procedures adopted by Fugro are in general accordance with ISO/DIS 19901-10:2018 (ISO, 2018) and ISSMGE (2005). The scope of marine geophysical investigation is according to agreed project specifications.

A marine geophysical investigation is commonly followed by marine soil investigation (ISO, 2014).

Apparatus

Overview

Reflection seismic survey spreads comprise the following components:

- Survey platform;
- Source to produce a discrete and repeatable acoustic pulse;
- Receiver (array) to receive the reflected signals;
- Positioning equipment;
- Recording/display unit(s).

Survey Platforms

Examples of survey platforms are:

- Geophysical survey (surface) vessel;
- Autonomous surface vessel (ASV);
- Remotely operated vehicle (ROV);
- Autonomous underwater vehicle (AUV).

Selection of a survey platform depends on factors such as scope of investigation, water depth and seafloor terrain (including gradients).

Sources

Acoustic sources are commonly classified on the basis of power levels and operation frequency. In general, higher power levels

result in lower frequencies and imply greater penetration in the seabed but with less resolution. Acoustic sources include:

- Pinger: a low energy device with an array of acoustic transducers that convert an electric pulse into an acoustic pulse within the frequency range of 3 kHz to 7 kHz;
- Chirp: type of pinger that emits a frequency-modulated pulse over a specified range of frequencies;
- Boomer: source powered by a rapid high-voltage discharge of capacitor banks through a spark gap. Combinations of energy supply and energy discharge units give available power levels ranging from 100 Joules to more than 1 000 Joules with a frequency range of 50 Hz to a few kHz;
- Sparker: a seismic source that operates by producing an electric discharge in water. The heat generated by the discharge vaporises the water at the electrode tips, creating an effect equivalent to a small explosion of bubbles, which oscillate and collapse after a few milliseconds. Sparker sources are generally limited to an energy range of 200 Joules to 10 000 Joules, though there is generally little advantage to be gained in increasing the energy level beyond 3 000 Joules;
- Airgun: source which injects a bubble of highly compressed air into the water.

In some systems, a source is combined with a receiver in a single unit (transducer), e.g. a pinger system mounted in the hull of a survey platform.

In many cases, a source is towed behind a geophysical survey vessel, either at the sea surface or at some depth below the sea surface.

Receivers

Receivers convert the transmitted acoustic energy (i.e. seabed reflections) into electric signals. Receivers include:

- Hydrophones;
- Geophones;
- Accelerometers.

Receiver configurations can comprise a single receiver (single channel), multiple receivers (ocean bottom nodes/ cables) or a group of receivers (array; multichannel). A group of receivers is incorporated in a flexible streamer, which is towed by the survey platform, either at the sea surface or at some depth below the sea surface.

Positioning Equipment

Positioning equipment typically includes multiple GNSS systems, mounted on the survey platform and in some cases on the source and receiver systems. For example, a streamer may be fitted with a tail buoy that includes a GNSS positioning system.

AUV and ROV survey platforms include subsea positioning systems.

Recording/ Display Units

A control unit is an integrated source trigger and recorder unit, which records the electrical signals from the receivers. The control unit may allow initial processing to the recorded data including:

- Overall and time varied gain (TVG);
- Swell filters;
- trace stacking;
- band-pass filters.

Signals are displayed in real-time and recorded digitally.

Procedure

Overview

The procedure for marine reflection seismics generally includes activities ranging from survey design to integrated geosciences. The agreed project specifications can cover some or all of the steps in the procedure.

Survey Design and Selection of Apparatus

Survey design depends on the required level of seabed detail. Particularly, it includes selection of apparatus and layout of survey track lines.

One or more seismic reflection methods can be selected, for example a combination of SBP and UHR methods. This selection phase takes account of factors such as:

- Feasible combinations of required resolution and penetration depth into the seabed;
- Inherent limitations, e.g. detailed imaging of the upper 0.5 m below seafloor (Peuchen and Westgate, 2018); this depth interval can be important for design of sub-sea templates, surface-laid pipelines and cables.

Survey platform, source and receiver (single or multiple) configurations are then selected. In general, such selection depends on expected wave heights, (tidal) currents, water depths, seabed conditions and sustainability considerations.

Where applicable, the tow depths of a source and of a receiver array (streamer) can be selected to minimise metocean effects (e.g. swell) on data quality of the recorded signals.

The survey line plan (i.e. survey design) typically considers:

- Required spatial detail of the ground model as function of survey track line spacing, e.g. lateral delineation of soil provinces, ground units and geological features;
- Detection of isolated seabed features, smaller than the distance between survey track lines (e.g. isolated boulders, shallow gas pockets, anthropogenic debris below seafloor etc.);
- Survey line orientations, taking into account seafloor morphological features that may impair adequate transmittal of seismic waves (e.g. survey track lines perpendicular to steep slopes to reduce unwanted noise in seismic data);
- Sustainability considerations including use of energy (e.g. reduction of negative effects on marine life).

Equipment Deployment

Equipment can be deployed (1) as integral part of the survey platform, (2) launched and towed behind a surface vessel and (3) by detached operations (e.g. for ocean bottom nodes).

For AUV and ASV operations, the survey line plan is uploaded in an internal navigation computer of these survey platforms. Following deployment, these systems autonomously acquire data along a programmed line plan, using a suite of acoustic sensors to avoid obstacles and maintain a specified speed and (in case of AUV) a near-constant elevation above seafloor.

For ROV operations, a ROV operator controls the survey platform from a vessel or from an onshore remote operations centre.

Equipment deployment can include a verification and testing programme for key components of the apparatus, for example by data acquisition along a line within the site. This line can be surveyed several times to adjust acquisition parameters (e.g. operating frequency and/or energy levels, equipment tow depth)

for local conditions to optimise trade-off between resolution and depth of penetration.

Data Processing

SBP data can be processed either in real-time or separately as a post-acquisition exercise. Basic processing steps include:

- Adding detailed geodetic positions to the acquired seismic traces;
- Applying a correction for tidal effects during data acquisition.

Additional processing typically applies to HR, UHR and UUHR seismic reflection data. Processing steps will vary according to agreed project specifications and site-specific conditions. The focus of the processing is typically on maintaining the relative amplitude relationships and preserving the high frequency content for the depth zone(s) of interest.

Processing techniques include (ISO, 2018):

- Gain recovery/amplitude manipulation;
- Velocity analysis;
- Mute;
- Normal moveout (NMO) correction and common mid-point (CMP) stack;
- Designature techniques;
- Demultiple techniques;
- Deconvolution before stack (DBS);
- Pre-stack time migration (PSTM);
- Post-stack coherent noise attenuation;
- Zero-phase conversion;
- Time varying filter (TVF);
- Time-depth conversion.

Assessment of Data Quality

For surface vessels, the quality of the data can depend on sea state, e.g. wave height during data acquisition. An example would be 'tugging' of a towing cable caused by large waves, having a detrimental effect on data quality.

Data acquisition in shallow coastal water may be impaired by surface breaking waves (i.e. surf zone). Acquisition in such areas is challenging and may require an exceptionally calm sea state.

Shallow gas in the seabed can make it impossible for seismic reflection methods to obtain adequate resolution at and below the upper surface of shallow gas. The presence of even a small amount of gas in the seabed causes substantial soil can drastically increase the attenuation of the acoustic signal, causing (acoustic) blanking.

Information from the data interpretation phase and the integrated geosciences phase can support assessment of data quality.

Data Interpretation

The data interpretation phase can make use of specialist interpretation software, allowing interrogation of seismic records or lines. Seismic reflectors, that correspond to geological and/or geotechnical ground units can be mapped as horizons. Furthermore, data interpretation allows mapping and assessment of potential geohazards, for example:

- Cemented coarse soil layers;
- Shallow gas;
- Buried channels (i.e. spatial variation in soil conditions).

Reflectors mapped in the time domain (e.g. two-way travel time as depth scale) can be converted to depth using an estimate for seismic velocity. Velocity estimates can be obtained from general

knowledge of the seabed, nearby marine geophysical investigation, a (seismic) velocity model and, preferably, by correlation (ground truthing) with site-specific geotechnical data (e.g. cone penetration test data showing distinct changes in cone resistance at soil unit boundaries) and data derived from borehole geophysical logging. Ground truthing increases reliability of data interpretation.

Integrated Geosciences

Integration of seismic reflection data with other methods of investigation can be done at different levels:

- Stratigraphic Zonation: integration focussing on achieving stratigraphic alignment between seismic profiles obtained by non-intrusive techniques and stratigraphic interpretation from results of intrusive ground investigation;
- Geotechnical Zonation: integration providing a vertical and horizontal zonation and includes mapping of 'soil provinces', with each soil province having a representative vertical soil profile and distinct seismic character;
- Geotechnical Zonation and Analysis – integration as per Geotechnical Zonation, additionally including engineering assessments for specific project requirements.

Results

Reporting Scope and Format

The results of a marine reflection seismics typically include documentation of data acquisition, data processing steps and interpretive features (e.g. seismic data character of the seabed and illustrative data examples). In addition, the following can be provided, depending on agreed project specifications:

- Track charts presenting positions of the survey track lines;
- Shallow structure maps, presenting depth to top or base of interpreted ground units;
- Isopach maps, presenting thickness range of interpreted ground units;
- Geological features maps, presenting positions of e.g. interpreted buried channels, shallow gas anomalies, etc.
- Cross sections;
- GIS data base;
- Digital, seismic interpretation project (e.g. Kingdom Suite project, Petrel project);
- Voxel ground model for GIS-type visualisation.

Reflection Amplitude

Reflection amplitude (high, medium, low) is the deviation of a seismic wave from the zero-crossing along a trace. It provides a measure of relative reflection strength and can provide an indication of lithological contrast (seismic velocity - density of ground), ground layer spacing, possible pore fluid/gas content etc.

In general, high amplitude events correlate to strong vertical variation of contrasting lithologies (e.g. sand / clay), while low amplitude indicates more comparable lithologies on both sides of an interface (e.g. sand / silty sand).

High amplitudes typically occur at seafloor, where there is a large contrast in density (unit weight) and velocity between seawater and seabed. A boundary between soil and rock is generally also associated with high amplitude reflectors. High amplitude reflections can also be associated with soil including (free, undissolved) gas, due to the low velocities for gassy soil.

A reversed (polarity) amplitude – as in "polarity reversal" – is an amplitude feature characterised by reversal of the seismic

reflection polarity along a horizontal interface, when followed from trace to trace. This may be due to a change in fluid/gas contents of pore space. Reversed polarity is commonly associated with gas in soil.

Seismic Frequency

Seismic frequency (high, medium, low) is commonly associated with resolution of seismic data. Resolution is the ability to distinguish two seabed features from one another (e.g. top and base of a ground layer and interbedding).

The vertical resolution (limit of separability) can be taken as $\frac{1}{4}$ of the seismic wavelength (λ). The wavelength in turn can be calculated from seismic velocity v , i.e. $\lambda = v/F$ where F is seismic frequency.

Seismic frequency decreases with increasing depth, as high frequencies in a seismic signal are more quickly attenuated. Conversely, seismic velocity generally increases with depth. As a result, vertical resolution of seismic data is depth dependent (Brown, 2004).

In general, high frequency seismic data imply reduced signal penetration below seafloor, but with greater overall vertical resolution.

Reflection Configuration

Reflection configuration can be described as parallel or sub-parallel, divergent, wavy, hummocky, chaotic, transparent etc.

Reflection configuration is related to the geometry/ pattern of ground layering/ bedding pattern resulting from specific depositional processes, original palaeotopography and post-depositional processes. For example, a high energy depositional environment (e.g. fluvial environment, debris flows) generally results in a more chaotic seismic response.

Reflection Continuity

Reflection continuity (continuous, discontinuous, truncation of reflectors) describes continuity of ground layers. It is directly related to sedimentation processes and post-sedimentation processes (e.g. erosion) and hence, to geological setting.

In general, the less energetic the environment, the more continuous the seismic response. Continuity also indicates greater lateral extent of certain depositional conditions, while discontinuities may suggest rapid changes in energy levels during deposition and/or effects of post-depositional erosion.

For example, flood plain deposits are generally associated with a low energy environment that may be laterally continuous. The reflections associated with a flood plain are generally continuous (parallel bedded reflectors). The reflections associated with a (meandering) stream or fluvial channel are localised and of low lateral continuity.

Multiple Reflections

'Multiples' are commonly associated with seismic datasets. Multiple reflections are reflections generated by acoustic waves travelling several times (reverberation) between two strong interfaces (e.g. seafloor and a boundary between rock / soil layers) before they arrive at the receiver. Multiple reflections can be strong and can interfere with reflections of geological interfaces, whereby the latter are obscured in the depth interval where the multiples appear.

For example, an acoustic wave traveling from its source may be reflected off the seafloor, the sea surface and the seafloor again before arriving at the receiver (seafloor multiples). The "ray-path"

of this multiple is approximately twice that of a normal source-seafloor-receiver ray-path.

If two strong subsurface reflectors are present, acoustic signals may “bounce” several times between them, before the wave-front can be recorded at the receiver (peg-leg multiples).

Seismic Artefacts

Seismic artefacts may be caused by lateral variation of seabed conditions, including buried boulders and steeply dipping ground strata. Examples are:

- Pull-up effects below seafloor sand waves, where seismic reflectors can appear at slightly higher elevations than where sand waves are not present. This is due to small seismic velocity variations and steep-sided slopes of the sand waves;
- Acoustic transparency and signal attenuation below crests of sand waves and enhanced amplitude reflections or signal tuning at the troughs of the sand waves. These feature can mask seismic signal;
- Pull-down effects as indicator for the presence of free gas in the seabed. Gassy soils have lower seismic velocity and hence acoustic waves travel slower than in fluid-saturated soils, causing seismic reflectors to (locally) occur lower than in non-gassy soils;
- Hyperbole-shaped reflections (i.e. diffraction hyperbolae) originating from “point sources” in the seabed. Typical point sources are buried boulders, pipelines and cables.

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Site Characterisation

Introduction

Site characterisation may be defined as a fit-for-purpose model of a site. For the context of this document, site is defined as a geographical part of the earth at a particular time or period, that:

- Incorporates the surface of the earth and ground below this surface;
- Can include water and air above the ground surface;
- Can include man-made objects and structures in or connected to the ground.

Ground is defined as soil and rock, including made ground, pore fluid and pore gas.

The site model is fundamental to managing ground risks and optimizing opportunities. The model is a prediction and a reduction of reality and can be used to:

- Provide sound information with which to define and assess the suitability of a site for proposed facilities;
- Detect and assess the possible effects of geohazards and changes in seabed conditions with time;
- Choose parameter values for verification of limit states and to assess the feasibility of building/installing, operating, and/or decommissioning a structure.

The model has interpretive limits that typically depend on:

- Structure characteristics and project phase such as conceptual design, installation and structure re-assessment;
- Data selected and available at the time of study:
 - Stratigraphic schematisation, e.g. partial data coverage or detection limits of deployed tools and an interface between strata may be more gradual than indicated;

- Level of detail and accuracy in interpretation of geotechnical parameter values, such as test data, sample size, quality, and coverage;
- Public-domain information such as geological understanding;
- Data visualisation algorithms, e.g. for data contouring.

Other terms used in practice for (parts of) site characterisation include integrated study, integrated geosciences, desk study, and seabed characterisation.

Site characterisation can also refer to the activities required to create the model of the site (e.g. Evans, 2010; Peuchen, 2012). Ideally, site characterisation benefits from integration of multidisciplinary data (e.g. geotechnical and geophysical data integration).

This document focuses on offshore projects. Site characterisation is an integral part of offshore structure design and operation according to reliability principles covered by standards and codes of practice; for instance API (2011, 2014 and 2015), ASTM (2018), DNV GL (2017), RenewableUK (2013), CEN (2004 and 2011); ISO (2009, 2014, 2016a, 2016b, 2019a and 2019b).

Table 1 shows levels of integration that can be considered.

The terms seabed and seafloor are according to ISO (2016a):

- Seabed comprises materials below the sea in which a structure is founded, whether of soils such as sand, silt or clay, cemented materials or, of rock
- Seafloor is defined as the interface between the sea and the seabed.

This document also uses the geological term 'sediments' as synonym of uncemented soil.

Table 1: Levels of Integration for Marine Site Characterisation

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 for 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.

Procedure

Figure 1 summarises the general procedure. Decisions on scope and adequacy are project-specific and depend on input such as project specifications, management of site risk, and schedule constraints.

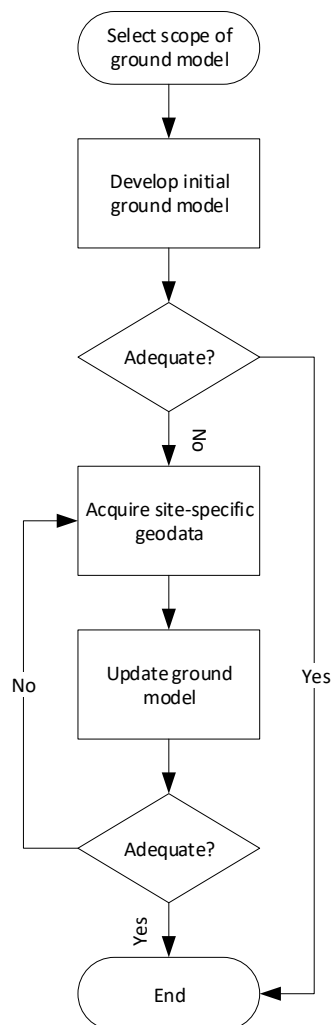


Figure 1: General procedure

Visualisation of Ground Model

Diagrams, still images, video and 3-D visualisation can communicate and convey site characterisation to experts, users of the model, and the public at large.

For Integration Levels 3 and 4 (Table 1), a common approach for data management and imaging is combining a GIS (geographical information system) with one or more separate software packages. IHS Markit® Kingdom software for geophysical data and GeODin® software for geotechnical data (e.g. Noll et al., 2010) are examples of such software packages for nearshore and offshore projects.

Site Hazards

Types of Hazards, Risk and Mitigation

Site hazards may be grouped into:

- Natural geohazards;
- Man-made hazards.

Natural geohazards are commonly referred to as geohazards or geological hazards. They are about past geological processes and events that have shaped the seafloor and seabed. Some of these processes may still be active today. The resulting seafloor topography, and geological and geotechnical conditions within the seabed can be hazardous when installing offshore structures including infrastructure (e.g. Clayton and Power, 2002; IOGP, 2009, 2017; API, 2011).

Man-made hazards include shipwrecks, fallen objects, seafloor debris and unexploded ordnance. Within the context of this document, man-made hazards exclude accidental events such as vessel impact, sabotage, well drilling problems, and fishing activities.

In relation to offshore activities, geohazards can be defined as local and/or regional site and soil conditions having a potential of developing into a condition (e.g. irregular seafloor topography) or process (e.g. currents, submarine slides) that could cause loss of life or damage to health, environments and/or assets. The event-triggering sources can be ongoing geological processes or human induced changes (IOGP, 2009). Figure 2 presents a schematic overview of offshore geohazards.

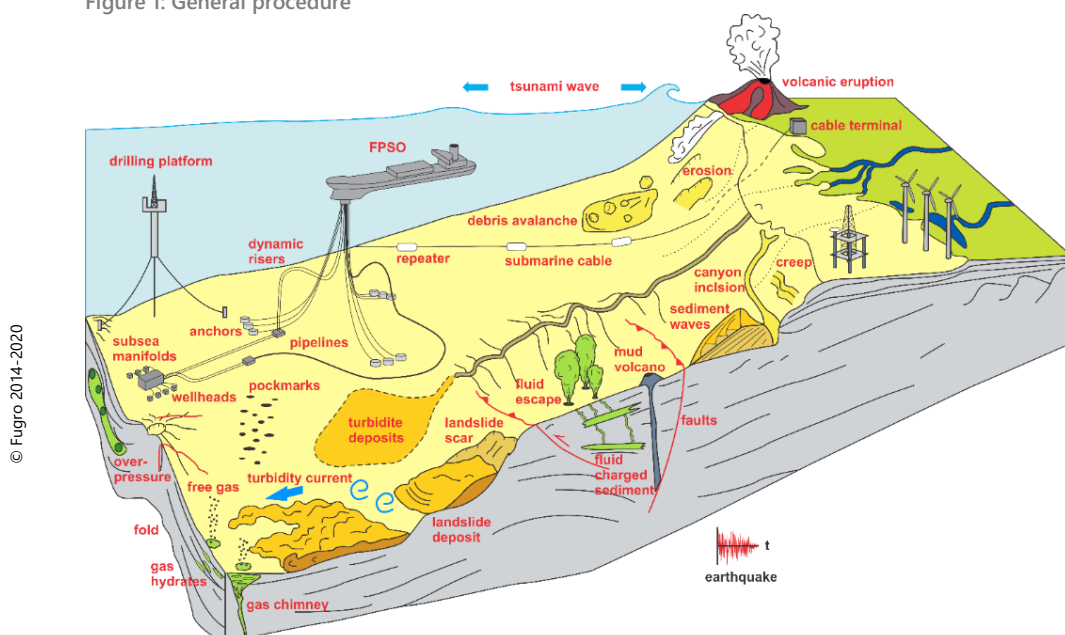


Figure 2: Offshore natural geohazards (modified after Campbell et al., 1986)

The damage potential of site hazards can range from, for example, local effects on pipelines and subsea structures to complete loss of all installations in a license areas and third party losses (IOGP, 2009).

The table below presents an overview of potential impacts and/or consequence associated with natural geohazards (and man-made hazards) occurring offshore.

Table 2: Potential Impact/Consequence Associated with Site Hazards

Impact/Consequence	Natural Geohazards and Man-made Hazards														
	Irregular Seafloor Topography	Seafloor Bedforms	Seafloor Outcrops and Hard Seafloor	Soil Liquefaction	Shallow Gas and Gassy Soils	Gas Hydrates	Gas and Fluid Seepage	Diapirs (e.g. Mud /Salt) and Mud Volcanoes	Earthquakes	Faults	Tsunamis	Slope Failure	Submarine Mass Movement	Wind, Waves and Currents	Seafloor Scour and Sediment Mobility
Uneven support (foundation instability)		x				x				x	x				x
Loss of support (structural stresses)				x			x		x		x	x	x		
Spanning (pipeline and flowlines)	x	x	x							x					
Increased foundation settlements, reduced access				x	x										
Burial/embedment leading to additional loading and reduced access		x		x									x		x
Reduced soil strength and bearing resistance				x	x		x								
Lateral loading of structure leading to overstressing of foundation/structure components									x		x	x	x	x	x
Structure displacement and structural damage				x					x	x	x	x	x		x
Increased potential for soil liquefaction					x	x	x		x		x			x	
Increased potential for shallow soil instability and submarine sliding					x	x	x	x	x		x			x	x
Foundation and structure installation difficulties	x	x	x		x	x	x								x
Steel abrasion, gouging and denting; excessive wear of trenching equipment			x												
Gas and fluid migration (excess pore pressures)					x	x	x	x		x	x			x	
Corrosion of steel structures, pipelines, flowlines					x		x	x							
Well (borehole) instability					x	x	x			x					
Mud losses (well/borehole drilling)										x					
Damage to casing string and pile foundations										x					
Presence of environmentally protected chemosynthetic communities					x		x	x							
Explosions leading to changed site conditions															x

Site hazards can generally not be treated on a statistical basis applying solely historical data. The nature of a hazard is often site and time dependent. In addition, natural geohazards are often interrelated. This may be due to a common trigger mechanism (e.g. earthquake, slope failure), or that one geohazard occurrence or process forms a trigger for other geohazards.

For instance:

- Earthquakes will induce dynamic actions on a structure and may induce elevated pore pressures leading to increased susceptibility to soil liquefaction;

- Slope failures and their deposits may result in irregular seafloor topography;
- Mud and salt diapirs are commonly associated with radial fault patterns, and continuous diapirism may result in (shallow) slope failures.

Table 3 highlights some relations between natural geohazards.

Table 3: Related Offshore Natural Geohazards

	Irregular Seafloor Topography	Seafloor Bedforms	Seafloor Outcrops and Hard Seafloor	Soil Liquefaction	Shallow Gas and Gassy Soils	Gas Hydrates	Gas and Fluid Seepage	Diapirs (e.g. Mud /Salt) and Mud Volcanoes	Earthquakes	Faults	Tsunamis	Slope Failure	Submarine Mass Movement	Wind, Waves and Currents	Seafloor Scour and Sediment Mobility
Irregular Seafloor Topography		x	x							x		x	x	x	x
Seafloor Bedforms	x													x	x
Seafloor Outcrops and Hard Seafloor	x				x		x	x				x			x
Soil Liquefaction					x	x	x	x	x					x	
Shallow Gas and Gassy Soils			x	x		x	x	x		x		x	x		
Gas Hydrates				x	x		x					x	x		
Gas and Fluid Seepage			x	x	x	x		x		x		x	x		
Diapirs (e.g. Mud /Salt) and Mud Volcanoes			x	x	x		x			x		x			
Earthquakes				x						x	x	x	x		
Faults	x				x		x	x	x		x	x	x		
Tsunamis									x	x		x	x	x	x
Slope Failure	x		x		x	x	x	x	x	x	x		x	x	x
Submarine Mass Movement	x				x	x	x		x	x	x	x		x	x
Wind, Waves and Currents	x	x		x							x	x	x		x
Seafloor Scour and Sediment Mobility	x	x	x								x	x	x	x	

Assessment of hazard probability of occurrence and frequency can be based on geomechanical modelling taking into account uncertainty in modelling of site conditions, soil parameter values, ongoing geological processes, actions and applied analysis methods (Clayton and Power, 2002; IOGP, 2009).

The risk of a site hazard is the sum of the product of the probability of a hazard event affecting a structure and damage consequence. The damage consequence can depend on factors such as structure robustness and vulnerability. The information in this document covers the nature of hazards and their potential implications, not the risk. Power et al. (2005) and Galavazi et al. (2006) describe risk analysis methodology.

Risk mitigation can include avoidance (e.g. a certain standoff distance to avoid structure interaction) and design for robustness.

Irregular Seafloor

Seafloor morphology can be irregular as a result of past or present geological processes. Human activities can also affect the seafloor topography. Irregular seafloor may be caused by (or be associated with) a number of natural and man-made phenomena. These include:

- Canyons and channels;
- Boulders (e.g. drop stones);
- Spudcan footprints;
- Anchor scars;
- Trawl marks and scars;
- Drill cuttings.

The scale of morphological features varies (e.g. scour marks, submarine canyons). The impact can differ per structure type and geometry.

Seabed Scour and Sediment Mobility

Seabed scour relates to the erosion of seabed sediments. Such erosion can occur under normal metocean conditions or can be enhanced as a result of a structure or multiple structures interrupting a natural flow regime above seafloor, thereby increasing flow velocities. Scour can be enhanced or initiated by secondary processes such as rocking of a structure.

Especially non-cohesive sandy (and silty) sediments are susceptible to scour. Erosion and transport of fine sand can start at a flow velocity in excess of 0.2 m/s. Local scour pits (or scour holes) can form shortly after installation of a structure. Their dimensions will usually vary in time depending on the flow regime.

Scour can occur in any water depth (from shoreline to deep sea). The flow regime due to wave- and tidal influence is generally stronger in shallow water than in deep water (Soulsby, 1997; Sumer and Fredsoe, 2002). In general, tide- and wave action, in combination with fluvial discharge of fresh water, determine the natural flow regime in coastal areas. Deepwater bottom current activity may result from density differences between water

masses and from global thermohaline ocean circulation. Resulting sedimentary accumulations are known as contourite drifts (Faugeres et al., 1999).

Seafloor variation can usually be characterized as some combination of the following (Whitehouse, 1998):

- Local scour and sedimentation; usually a steep-sided scour pit around a structure or structural element;
- Global (or general) scour; a (shallow) scoured basin of large extent around a structure, possibly due to overall structure effects, multiple structure interaction, or wave-soil-structure interaction;
- Overall seabed movement; erosion, deposition, bedform migration that would also occur in the absence of a structure (i.e. regional scour).

Seafloor Bedforms

A seafloor bedform is a morphological feature formed by interaction of wave action and (tidal) currents and cohesionless sediment (i.e. sand/silt). Bedforms are typically found in sandy areas at a continental shelf.

A characteristic of bedforms is their mobility (Table 4). Sand waves tend to move slowly (metres per year) or flex their crests with (tidal) currents. Smaller-scale ripples tend to be more mobile, in the order of metres per day.

Table 4: Seafloor Bedforms

Bedform Type	Related Flow	Wavelength [m]	Amplitude [m]	Time-scale	Migration Rate	Source
Ripple	Instant flow	0.1 to 1	0.01 to 0.1	Hours	> 1 m/day	Morelissen et al. (2013) Reineck and Singh (1980) Dodd et al. (2003)
Megaripple	Storm surges	10 to 20	0.1 to 1.5	Days	100 m/year	Morelissen et al. (2013) Ashley (1990)
Sand wave	Tidal currents	50 to 1000	2 to 18	Decades	1 m/year to 10 m/year	Morelissen et al. (2013) Ashley (1990) Reineck and Singh (1980) Dodd et al. (2003)
Long bed wave	Unknown	1500 to 2500	1 to 5	Unknown	Unknown	Morelissen et al. (2013) Reineck and Singh (1980) Dodd et al. (2003)
Sand bank	Tidal currents	5000 to 10000	5 to 50	Centuries	m/year	Morelissen et al. (2013) Reineck and Singh (1980) Dodd et al. (2003)

For structure design it is important to know which part of the seabed and/or the bedforms is actually mobile. For example, cable trenching can modify bedforms. The rate at which the bedforms recover after cable trenching will depend on sediment transport rate and supply of sediment.

Seafloor Outcrops and Hard Seafloor

Seafloor outcrops and hard seafloor ground conditions commonly include:

- Shell and coral banks, reefs, which are common in shallow waters in the tropical zones;
- Local patches of cemented soil (e.g. hard ground, cap rock). Examples are authigenic carbonates around pockmarks, Kurkar ridges (cemented aeolian dunes) in the eastern Mediterranean Sea, beach rocks (cemented beach sediments) in the Caribbean Sea, sabkha deposits (evaporitic-tidal floodplain deposits) in the Arabian/Persian Gulf and Gulf of Suez;

- Crust composed of precipitated metal sulphides associated with hydrothermal activity (e.g. black and white smokers) in vicinity of tectonic plate boundaries and faults;
- Outcrops of rock. Examples are pre-Quaternary sand- and limestone beds offshore West Africa, sedimentary and metamorphic rocks exposed in the Irish Sea.

It should be noted that seafloor outcrops and hard seafloor may have environmental protection status or legislative implications.

Cementation of soil may result from sub-marine cementation processes. Cementation may also have resulted from past sub-aerial exposure of a continental shelf during low sea level stands under arid climate conditions. Cementation generally occurs in carbonate-rich and hyper-saline environments.

Diapirs and Mud Volcanoes

A diapir is a domal upwelling of sediment, rock or salt that forms in response to tectonic forces, density differences, and high overburden pressures. Diapirs can pierce through a stratigraphic overburden and create an envelope of overconsolidated soils,

deformed rock, and sediments around a diapir core (e.g. salt). Generally, a circular dome-shaped topographic feature develops when a diapir approaches the seafloor. Diapirs are commonly associated with radial faulting patterns and locally increased seafloor slopes.

Salt diapirs are known to be present in, for example, the Gulf of Mexico, offshore Brazil and West Africa, and the North Sea.

Mud diapirs and mud volcanoes are usually associated with rapidly-deposited sediments and in situ pore pressure conditions significantly higher than hydrostatic (overpressured). Additionally, high vertical and horizontal stresses typically apply, caused by faulting, folding and uplift processes.

Mud diapirs and mud volcanoes occur mostly in (historic) delta areas: Nile Delta (offshore Egypt), Absheron Ridge (offshore Azerbaijan, Caspian Sea), Makran Ridge (offshore Iran, Arabian Sea), and Niger Delta (offshore Nigeria).

Release of pressure is commonly provided by faults and folding of the strata. Sediments mixed with over-pressured fluid and gas (mud) migrate upward through the stratigraphic overburden in vertical columnar zones (diapirs). Usually the over-pressured muds enter fault planes, thus causing diapirism along faults. A mud volcano can form when a mud diapir breaks the seafloor.

In general, mud volcanoes are conical, as tall as 65 m and up to 2 km across. The size and shape of a mud volcano depends on the frequency of expulsion and the type of material ejected. This can be unconsolidated soils, overconsolidated material, fractured rock (e.g. breccia), oil, gas, and water (Snead, 1972; Newton et al., 1980; Delisle et al., 2002; Delisle, 2004; Delisle, 2005). Not all offshore mud volcanoes are active. Eruptions are believed to be episodic.

Shallow Gas and Gassy Soils

Gas may be present (trapped) in the seabed (e.g. gassy soils). Shallow gas can comprise a mixture of different gases, such as carbon dioxide, hydrogen sulphide, ethane, and methane. In general, the gases originate from bacterial decay of organic matter (biogenic gases) within a few metres of the seafloor. Gas may also come from sources much deeper in the stratigraphy and migrates upwards through pores and cracks in the soil and rock (petrogenic gases).

Shallow gas may be present dissolved in pore water, as free gas in gas-filled voids or bubbles, and as gas hydrates. Over time, gas in soil may increase the in situ pore pressures and result in excess pore pressures.

Migration of gas in soil can result in accumulation of gas in the seabed below a foundation. Shallow gas in the pore water can have a serious effect on foundation behaviour.

In addition, shallow gas can be toxic to humans, can combust and explode.

Soil property measurements on geotechnical samples containing shallow gas may not be representative of in situ properties.

Gas Hydrates

Gas hydrates are ice-like crystalline solids composed of water molecules surrounding a molecule of gas, generally methane. Gas hydrates can only form when gas is over-saturated in water. Gas hydrates are stable under high pressure and low temperature conditions, and may be present at seafloor and in shallow sediments, generally in deep water environments in excess of 500 m below Mean Sea Level (Rastogi et al., 1999; Von Rad et al., 2000).

Stable gas hydrate acts as cement and increases strength and rigidity of soil.

Natural gas hydrates are regarded as a geohazard when they dissociate, start 'melting'. Both water and gas are released into soil when gas hydrates dissociate. This can result in formation of 'gassy soils'. The addition of water and gas may decrease soil strength and form a weak layer (Orange and Breen, 1992; Judd and Hovland, 2007). Gas hydrate dissociation may be initiated by human activities, e.g. flow of 'hot' hydrocarbons through well production casings, pipelines and flowlines.

Gas hydrates may form as a result of human activity. Gas hydrates can be a by-product of hydrocarbon production, forming hydrate plugs in the wellbore, around leaking joints and in pipelines. If a deep water exploration or production well is leaking, gas introduced into the shallow soils may react with water molecules to form hydrate layers or nodules.

Gas and Fluid Seepage

Gas and fluid seepage at seafloor is commonly associated with pockmarks. Pockmarks are roughly circular or conical depressions in the seafloor, generally 1 m to 350 m wide and up to 35 m deep (Newton et al., 1980; Von Rad et al., 2000; Judd and Hovland, 2007).

Pockmarks form by disruption of a pore pressure environment. This disruption may be triggered by natural or human causes, and can form on time scales of less than a year. Pockmarks can be intermittently active over long periods of time or can grow with explosive eruption events. The sediments in a pockmark are generally variable and may be overconsolidated.

When gas seeps continue over a long period of time, biological processes may cause cementation of the seabed sediments. Formation of authigenic carbonates can take place around the seeps (Judd and Hovland, 2007; Ding, 2008). In some cases, unique ecological habitats form in and around pockmarks. Such habitats may be protected by environmental legislation.

Authigenic carbonates may form thin crusts of weakly cemented sediments (hard grounds). They can be continuous over distances of several hundreds of metres (Von Rad et al., 2000). Locally more massive, competent layers of authigenic carbonates can be present as hard cemented layers or 'lenses'. They may form large build-ups and seafloor mounts (Judd and Hovland, 2007).

Apart from natural seeps, gas seepage may also be induced by drilling activities (e.g. geotechnical drilling, hydrocarbon exploration drilling). The drilling process may cause fracturing of soil and rock, when drilling mud pressures exceed the fracture pressure of the soil or rock (i.e. hydraulic fracturing). These fractures may form pathways for fluid and gas migration into the wellbore and up to seafloor. A wellbore or leaking well casing may form a pathway to the surrounding rock and soil formations, introducing gas into sand layers in the shallow subsurface. Overtime, the introduced gas may affect the geotechnical properties of a soil and have serious effects on foundation behaviour.

Drilling-induced fluid flows (e.g. shallow water flows) occur when a pressurised sand body (aquifer) encapsulated in clay is penetrated by the drilling process. Shallow water flows are common offshore large river deltas, such as the Mississippi Delta (Gulf of Mexico) and the Nile Delta (offshore Egypt). The sand bodies are commonly derived from sediment deposition out of turbidity currents.

Earthquakes

An earthquake, or seismic event, occurs after stresses in the earth's crust that have gradually built up, are suddenly released by movements along a fault. The movement generates seismic waves which propagate away from the earthquake epicentre. Most earthquakes occur along tectonic plate boundaries.

The location, magnitude, and frequency (recurrence) of earthquakes cannot be reliably predicted. The probability of seismic events can be assessed on the basis of historic records of earthquake activity.

Seismic impact depends on geotechnical conditions at the site and structure design. Seismic activity may induce faulting, soil liquefaction, slope failure, and tsunamis.

Soil Liquefaction

Two types of liquefaction may be distinguished:

- Gravitational (sometimes called static- or flow-) liquefaction, usually occurring in submerged slopes;
- Cyclic liquefaction, usually generated through strong cyclic forces.

Soil liquefaction or cyclic mobility represents a decrease of soil strength and stiffness caused by an increase in pore water pressure in saturated soil. Soil liquefaction usually occurs in response to sudden change in stress condition, causing it to behave like a liquid. Examples of cyclic and dynamic actions include earthquake shaking, storm wave loading, structure displacements upon cyclic load application, pile installation by driving, and vortex vibrations due to fluid flow around a structure.

Liquefaction potential can be significant for loose cohesionless soils present close to ground surface (seafloor) and below the water table. Dense sands, loose unsaturated sands, and some sensitive cohesive materials can also liquefy under some conditions. In addition, the presence of gas in loose sands can change soil behaviour and may potentially cause liquefaction (Grozic, 2003).

Faults

A fault is a planar fracture or discontinuity in a volume of soil or rock along which significant vertical and/or horizontal displacement has occurred (Figure 3) (i.e. faulting). Fault zones are areas where multiple fractures and faults occur in close proximity, with similar moment direction.

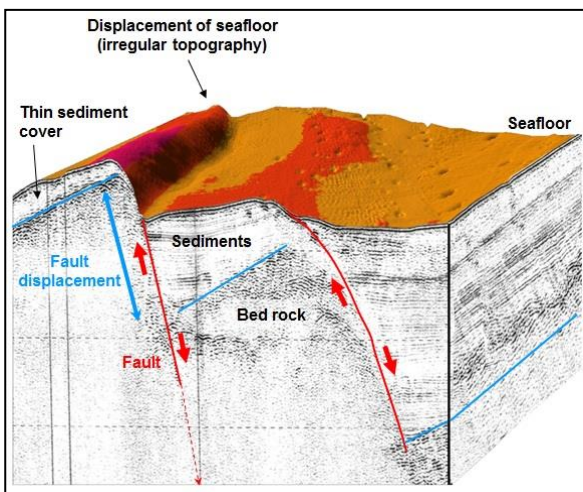


Figure 3: Surface and subsurface expression of fault displacement

Faults can be associated with:

- Tectonic activity (e.g. at tectonic plate boundaries, earthquake zones);
- Laterally variable soil subsidence and compaction;
- Soil contractions (e.g. polygonal faulting in North Sea and West African seabed sediments);
- Diapirism (e.g. radial faulting);
- Slope failure (e.g. headwall scarp, failure planes, tension cracks).

Movement along the fault plane (and hence soil displacement) is a semi-continuous process acting on time scales ranging from years to millions of years. Faults are commonly considered to be inactive if there has been no observed movement or evidence of seismic activity during the last 10,000 years. In this case a fault can be covered by a uniform layer of soil (i.e. without a clear discontinuity surface being present). Depending on crustal stresses and changes therein, apparently inactive faults may be reactivated causing further soil displacements and even seismic events.

Faults may result in a displaced, stepped seafloor and/or irregular linear topographic features on the seafloor (e.g., headwall scarps). In addition, stratigraphic sequences are displaced in the seabed.

Deep-seated faults, with lengths of hundreds to thousands of metres, may be associated with earthquakes. The build-up of stresses due to differential movement in the earth's crust may be released along these deep-seated faults, whereby large amounts of energy move through rock and soils in the form of pressure waves and shear waves. These deep-seated, earthquake-generating faults are sometimes referred to as seismic faults.

Tsunamis

A tsunami (or surge wave) is a series of ocean waves of long wave lengths which are created when a large volume of water is suddenly displaced by a submarine earthquake, landslide, or volcanic eruption (Figure 4). In the open ocean, tsunami waves travel at high speeds (in excess of 800 km/h) with heights of, say, less than 0.05 m. As they approach the coast the velocity decreases (to approximately 50 km/h) and the wave height increases up to several metres or tens of metres. At the coastline, the force of a tsunami wave can cause loss of life, damage to buildings and infrastructure, large scale erosion (scour) and flooding of low-lying areas.

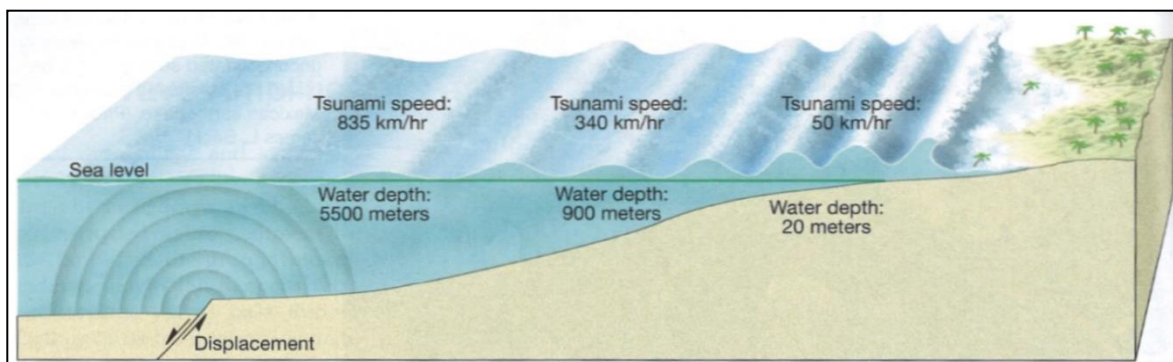


Figure 4: Tsunami generated by fault displacement offshore

Slope Failure

Slope failure occurs when downslope driving forces acting on seabed exceed resistance. In general, slope failure results in the down-slope movement of a soil mass (see section titled 'Submarine Mass Movements'). Slopes may be unstable at any water depth.

Slopes may develop due to tectonics, high sedimentation rates, or incision and erosion by seafloor currents and flows.

Slope failure can be triggered by earthquakes, strong currents, storms (wave actions), tsunamis, volcanism, and human activity (Hampton et al., 1996; Mulder and Cochonat, 1996; Locat and Lee, 2005; Judd and Hovland, 2007; Rogers and Goodbred, 2010).

Usually a combination of two or more factors influences slope failure, e.g. presence of shallow gas and an earthquake (Orange and Breen, 1992; Judd and Hovland, 2007). Slopes can be unstable due to low shear strength and overpressured strata (e.g. shallow gas). Seabed may fail on slight slopes as little as 0.5° (Hampton et al., 1996; Judd and Hovland, 2007).

Failure scarps and oversteepened slopes are commonly associated with past slope failures. Past slope failures may be reactivated if a trigger (e.g. pore pressure build-up, earthquake)

is present. The seafloor morphology resulting from a slope failure may be irregular and undulating (see section titled 'Irregular Seafloor Topography').

Submarine Mass Movements

A submarine mass movement is a displacement of seabed material driven directly by gravity or other body forces, rather than stresses associated with fluid flow. The deposits of submarine mass movements are commonly referred to as mass transport deposits (MTD).

Submarine mass movements commonly follow from slope failures and include the following processes (Figure 5) (Lee et al., 2007):

- Slides:
 - Translational slide
 - Rotational slide
- Mass flows:
 - Debris flow
 - Debris avalanche
 - Mud flow
 - Liquefaction flow
 - Turbidity current

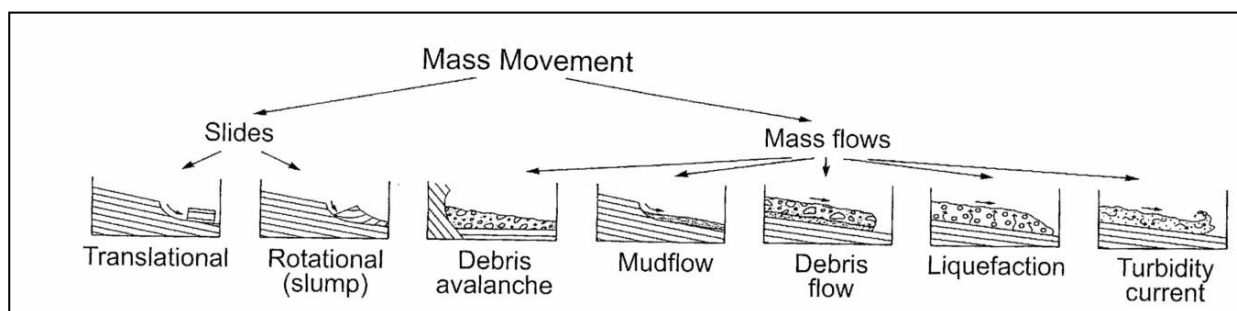


Figure 5: Submarine mass movement classification (after Lee et al., 2007)

Slides are movements of essentially rigid, undeformed masses along discrete failure/slip planes. If slip occurs along a planar surface the slide is referred to as a translational slide. If slip occurs along a curved failure plane and the rigid mass shows rotation, the slide is referred to as rotational.

If moving sediments take a form of viscous fluid, the feature is referred to as mass flow or gravity flow. Mass flow deposits show considerable internal deformation with many invisible or short-lived internal slip surfaces. Submarine slides can become mass flows as the failed material progressively disintegrates, gets entrained with surrounding water and moves downslope.

Debris flows are mass flows in which sediments are heterogeneous and may include larger clasts supported by a

fine-grained soil matrix. Mud flows involve predominantly fine-grained (mud) sediments. Turbidity currents involve downslope transport of a relatively dilute suspension of sediment grains that are supported by an upward component of fluid turbulence. Turbidity currents often evolve from disintegration and dilution of debris and mud flows. Liquefaction flows occur when loosely packed sandy sediments collapse under environmental conditions (e.g. cyclic actions by waves or earthquakes; see section titled Soil Liquefaction). Debris avalanches occur where slides collapse and disintegrate into smaller pieces. They move rapidly without following pre-existing channels or valleys.

The potential impact of submarine mass movements on a structure depends upon the location or orientation of the structure in relation to the movement direction (Figure 6).

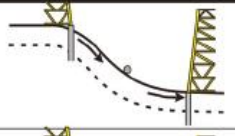
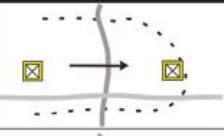
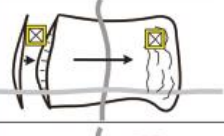
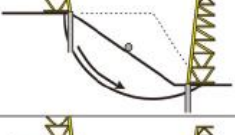
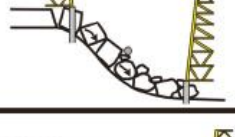
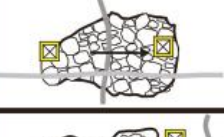
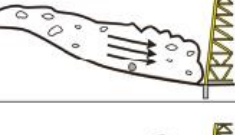
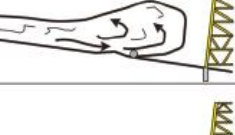
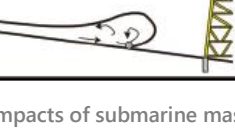
Mass Movement Mechanism	Impact on Foundations ☒		Impact on Pipeline/Flowline/Cable •		
	Profile View	Nature of Force on Foundation	Plan View	Orientation of Movement to Installation	
				Parallel	Perpendicular
Creep		Rotation About Base		Dragging Rupture Spanning	Dragging Rupture Spanning
Translational Slide		Translation Downdrag at Crest Uplift at Toe		Stretching at Crest Compression at Toe Loss of Support Rupture Spanning	Dragging Loss of Support Rupture Spanning
Rotational Slide		Rotation About Top Downdrag at Crest Uplift at Toe		Stretching at Crest & Toe Loss of Support Rupture Spanning	Dragging Loss of Support Rupture Spanning
Debris Avalanche		Translation/ Rotation +/- Downdrag +/- Uplift		Compression & Stretching Loss of Support Rupture Spanning, Burial	Dragging Loss of Support Rupture Spanning Burial
Debris Flow		Loading Burial Scour		Compression Burial Loading Scour	Dragging Burial Loading Scour
Liquefied Flow		Loading Burial Scour		Compression Burial Loading Scour	Dragging Burial Loading Scour
Fluidised Flow		Loading Burial Scour		Compression Burial Loading Scour	Dragging Burial Loading Scour
High Density Turbidity Current		Loading? Burial? Scour		Burial Loading Scour	Burial Loading Scour
Low Density Turbidity Current		Scour?		Scour	Scour

Figure 6: Potential impacts of submarine mass movements on platform foundation and pipeline (modified after Thomas et al., 2009)

Wind, Waves and Currents

Periods of extreme weather conditions, such as (tropical) storms, monsoons, peak wind, waves, and current regimes can cause lateral and cyclic actions on the seafloor and any seabed-supported structure. In addition, adverse weather conditions may complicate structure installation activities.

Peak wave and (seafloor/bottom) current regimes can also cause changes in seafloor conditions due to scour and burial (i.e. sediment remobilisation), winnowing of seafloor sediments (i.e. removal of fine/clay-size materials) and development of irregular seafloor topography.

Tidal variation and atmospheric pressure fluctuations as a result of storms are known to change pore pressure conditions in the seabed, potentially creating circumstances leading to soil failure and liquefaction.

Estimation of environmental actions is relatively inaccurate. It normally involves statistical data for a specific geographic region and various procedures for modelling the interaction of a structure and its environment.

Man-Made Hazards

Human activities and anthropogenic (i.e. man-made/man-induced) features, debris, or obstructions can have an adverse effect on an offshore structure.

Seafloor features and objects have been left by human activities since the dawn of mankind. Shipwrecks can form archaeological sites, war graves, enhance ecological diversity, and may be restricted areas.

In addition, offshore energy activities such as drilling, (jack-up) platform installation and decommissioning and resulting footprints may alter seafloor topography and/or potentially alter seabed conditions (e.g. drill spoils, gas charging as a result of gas migration along exploration wells).

Commonly encountered man-made hazards include:

- Unexploded ordnance (UXO);
- Existing energy facilities (e.g. fixed platforms, pipelines, manifolds, wellheads, power cables etc.);
- Telecommunication cables;
- Shipwrecks;
- Fallen objects (e.g. shipping containers).

These hazards can complicate structure installation and design if not identified at an early stage.

Activities such as hydrocarbon extraction and deep salt mining can change site conditions, for example causing regional subsidence of the seabed and/or trigger fault activity (Barton et al., 1987; Broughton et al., 1998; Broughton et al., 1997; Gebara et al., 2000). Subsidence can range from millimetres to tens of metres. It typically depends on reservoir size, mechanical properties of reservoir and overlying ground, reservoir depth, production rate, pressure drawdown, and duration.

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Geotechnical Analysis

Approach

A geotechnical design situation or a re-assessment of an existing structure requires geotechnical analysis, including evaluation of hazards and verification of relevant limit states. Geotechnical analysis follows design philosophies included in standards and codes of practice, where available. All consider that the resistance (or capacity) of a geotechnical system must be greater than the actions (demands or loads) on the system for an acceptable or required level of safety or reliability (ISO, 2015).

The approach for geotechnical analysis typically includes these steps:

- selection of procedures and models for geotechnical analysis
- processing and integration of geotechnical information, e.g. by preparation of geotechnical logs, cross sections, geographical information system GIS and/or 3D ground model
- site characterisation including hazard identification
- selection of geotechnical parameter values for calculation models
- application of calculation models and evaluation of results.

The approach for geotechnical analysis includes assumptions and premises. One premise is that the client's activities are state-of-the-practice in all areas, including planning, engineering, construction, operation and maintenance of a geotechnical system or structure.

Hazard Evaluation

Hazards are situations or events with potential to cause damage (ISO 2000; 2013). Hazard evaluation typically includes classification, estimation of probability of occurrence and measures for countering the hazard. Examples of hazards are abnormal environmental events, accidental events, geohazards and man-made site hazards. Note that event probability differs from risk, where risk is defined as the product of probability and consequence.

In many geotechnical situations, hazard evaluation will not be complete and exact. It will be necessary to draw on so-called tacit expert knowledge. This means senior expertise, with access to geotechnical knowledge and experience. Judgement and opinion are inevitable and a senior expert or a team of senior experts is more likely to arrive at a correct understanding and an appropriate way forward. Judgement is qualitative and subjective. Table 1 shows probability expressions intended for a context of approximate and subjective probability of the occurrence of a hazardous event during a defined exposure period.

Table 1: Expressions for Approximate and Subjective Probability (adapted from Peuchen et al., 2015)

Short Descriptor	Verbal Descriptor	Approximate Probability for Exposure Period
Negligible	Unlikely, although the possibility cannot be ruled out completely	0 to 0.01
Low	Not probable, although uncertain	0.01 to 0.1

Short Descriptor	Verbal Descriptor	Approximate Probability for Exposure Period
High	Credible, possibility can be described with reasonable confidence by known physical conditions or processes	0.1 to 1

Measures for countering a hazard include source elimination, avoidance, implementation of a barrier, minimising consequences and design for the hazard.

Limit States

Limit states may be grouped into Ultimate Limit States (ULS, for example for structure stability), Serviceability Limit States (SLS, for example for avoiding excessive settlement), Fatigue Limit States (FLS, for example for structural integrity of a pile) and Accidental Limit States (ALS, for example for impact of an object). Verification of a limit state usually involves one or more of the following approaches:

- calculation models
- prescriptive measures
- experimental models and load tests
- observational method.

Features of a calculation model include:

- method of analysis typically including simplifications and modification of the results where necessary to improve accuracy or to allow for uncertainty and systematic error
- actions, such as (a sequence of) imposed loads or imposed displacements
- geometrical data, such as the shape of a geotechnical structure, geometry of the ground surface, water levels and ground strata
- values of geotechnical parameters of ground (soil, rock, pore fluid, pore gas) and other materials
- limiting values of, for example, deformations and vibrations
- partial factors or safety factors.

The common analytical models rely on semi-empirical and direct methods of analysis.

Prescriptive measures generally involve (1) conventional and conservative details in the design and (2) attention to specification and control of materials, workmanship, protection and maintenance procedures. Their use is often applicable where calculation models are not available or not necessary. Examples are prescriptive measures for ensuring durability against chemical attack or frost action.

Experimental models and load tests can help to justify a design approach. Important considerations for evaluation of the results include differences in ground conditions, time effects and scale effects.

Prediction of geotechnical behaviour is often difficult. The observational method allows carefully planned monitoring during construction and includes planned contingency measures where necessary. Assessment of the monitoring results takes place at appropriate stages.

Design Philosophies

Design philosophies typically incorporate geotechnical calculation models and corresponding (partial) factors. These partial factors or safety factors may vary depending on the specific design scenario.

Design philosophies for the ULS may be grouped as follows:

1. Working Stress Design (WSD) or Allowable Stress Design (ASD).
2. Partial Factor Design (PFD) or Limit State Design (LSD).
 - a. Factored material properties.
 - b. Factored resistance.

The WSD method uses global safety factors applied to unfactored values (or ultimate values) of resistance.

The PFD methods the ULS use partial action factors and partial factors applied to resistance. The partial action factors are applied to unfactored values of actions. This results in design values for actions. The factored material properties and factored resistance methods differ by their calculation of resistance. The method for factored material properties applies partial material factors to unfactored values of material properties such as undrained shear strength of soil. The factored values are then used in the calculation model to obtain a design value for resistance (factored resistance). The factored resistance method uses unfactored values of material properties in the calculation model and then applies a partial resistance factor to obtain a design value for resistance. An additional factor γ_d can be considered to account for model uncertainty or other uncertainties not covered by other partial factors (ISO, 2013).

API RP 2A-WSD Fixed Offshore Platforms (API, 2014) is an example of the WSD approach for the ULS. Eurocode 7 Geotechnical Design (CEN, 2004; 2007), ISO Offshore Structures ISO 19900 and ISO 19901-4 (ISO 2013; 2016) and API RP 2GEO Geotechnical and Foundation Design Considerations (API, 2011) provide design principles according to the PFD approaches.

Design philosophies for the ALS, SLS and FLS are similar to the ULS. Global safety factors and partial factors will differ from the ULS.

Geotechnical Parameter Values

Design Process

Assignment of geotechnical parameter values or soil property values is according to the following steps:

1. Site characterisation and stratigraphic schematisation.
2. Evaluation of derived values of geotechnical parameters.
3. Selection of representative values of geotechnical parameters and application in a calculation model.

The selection of representative values of geotechnical parameters takes place within the context of a calculation model and thus includes consideration of limit states, actions, geometry, limiting values and partial factors or safety factors. Divorcing the selection of representative geotechnical values from the actual use and evaluation of a calculation model can lead to errors.

Stratigraphic Schematisation

General site characterisation is necessary before stratigraphic schematisation and before evaluation of the results of specific tests and observations. Such site characterisation comprises a general assessment of the character and basic constituents of the ground (soil and rock classification) and their possible change in time.

Typical parameters for soil classification include particle size distribution, water content, carbonate content, Atterberg limits, unit weight, relative density and undrained shear strength. Typical parameters for rock classification include mineralogy, water content, unit weight and uni-axial compressive strength.

The extent of stratigraphic schematisation depends on the nature of the actions, geometrical quantities of the structure that interacts with the ground, volume of ground that represents the domain of influence with respect to the limit state, spatial ground variability, simplification of ground conditions, e.g. undrained versus drained foundation response.

Two competing factors apply to spatial ground variability: (1) the spatial averaging of properties over a potential failure surface, which reduces the coefficient of variation of property values (i.e. with respect to that for the location under consideration) and (2) the tendency for a failure surface to follow the path of least resistance.

Stratigraphic schematisation can include evaluation of:

- basic parameters such as undrained shear strength and relative density;
- geological and hydro-geological setting;
- results of a geophysical survey;
- hazards such as potential instability of the ground;
- water levels;
- ground and ground water, with respect to structure durability.

Derived Values of Geotechnical Parameters

This document considers Eurocode 7 EN 1997-1:2004 for definition of *derived value*: "value of a geotechnical parameter obtained by theory, correlation or empiricism from test results". Where applicable, this document considers derived values to include measured values, test results, correlation values, theoretical values and empirical values. Borehole geophysical logging, in situ testing, laboratory test measurements and other relevant data provide a basis for obtaining derived values of geotechnical parameters.

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 strength by consideration of principal stress conditions and a theoretical deformation model.

Standards for in situ tests can require reporting of (normalised) derived values that can serve as direct input for some calculation models. An example is the use of CPT cone resistance for the calculation of axial pile resistance. A more common approach is to obtain derived values of geotechnical parameters from in situ tests on the basis of empiricism or (simplified) theory or a combination thereof.

This document considers *low estimate*, *best estimate* and *high estimate* values for derived values. The use of statistical methods can help to identify outliers and can support selection of best, low and high estimates. In statistical terms, this document considers a best estimate value as a mean value of data points acquired for a soil province, ground unit, stratum or multiple soil layers. Low and high estimates aim for the quantile associated with the 5 % fractile. Comments are as follows:

- Low, best and high estimates consider a reference method or procedure. This is because a test result or a derived value typically depends on the method(s) selected to obtain the parameter value. For example, undrained shear strength derived from a triaxial test on an intact soil specimen will

depend on the sampling method, sample handling practice, laboratory test procedure and whether undrained shear strength is derived from maximum deviator stress or maximum principal stress ratio.

- Low, best and high estimates can include judgement and opinion, particularly for a limited quantity or absence of test results and derived values. This implies that outliers may be ignored and that a bias may be introduced relative to the available data. Judgement and opinion consider physically credible values, comparison of data with results from other tests and *a priori* knowledge such as geological setting and comparable experience.
- A wide spread of data can indicate spatial variability of soil. This means that averaging of test results and derived values can obscure a weaker or stronger zone.
- A calculation model usually requires specific schematisation of soil stratigraphy and model-specific selection of parameter values. This is not covered by low, best and high estimates.

Representative Values of Geotechnical Parameters

Industry uses multiple terms for unfactored values of geotechnical parameters that are applied in a calculation model, for example:

- CEN (2004) considers “characteristic value”, representing a cautious estimate for the value affecting the occurrence of a limit state;
- DNV GL (2017a and 2017b) consider “characteristic value”, generally a low value with a prescribed probability of being favourably exceeded; sub-groups are “best estimate”, “lower bound” and “upper bound”;
- ISO (2013) considers “representative value” for a parameter used in a calculation model, with sub-groups “characteristic value” and “nominal value”; nominal value is defined as a value assigned to a basic variable determined on a non-statistical basis, typically from acquired experience or physical conditions;
- ISO (2015) considers “characteristic value” and “nominal value, with emphasis on characteristic value defined as “value specified preferably on statistical bases, so it can be considered to have a prescribed probability of not being exceeded”. A footnote to the definition of characteristic value refers to a nominal value that may be specified in cases where a statistical distribution is not known.

The text below uses *representative value* according to ISO (2013) for the value of a geotechnical parameter required for a calculation model.

The selection of a representative value takes account of probable differences between derived values of a geotechnical parameter and the geotechnical parameter that actually affects the behaviour of a geotechnical structure. In most cases, this is because no reliable and affordable geotechnical methods are available for direct and accurate representative values required for a calculation model. In other words, (multiple) test methods for derived values are typically selected based on feasibility in terms of available technology, economics and schedule. This limits applicability for the selection of representative values. For example, in situ relative density is defined relative to in situ void ratio and index void ratios. In situ void ratio can be obtained from laboratory tests on undisturbed, intact soil specimens. This is technically feasible for a marine setting, but considerations for economics and schedule lead to substitute, approximate, CPT-based correlations for relative density and their associated statistical distributions.

Other reasons for differences between derived values and representative values can include inhomogeneity of the ground, extent of the zone governing a particular limit state, uncertainties in geometrical data and analytical model, time effects, brittle or ductile response of the ground, influence of construction activities.

Representative values can be lower values, which are less than the most probable value, or upper values, which are greater. Each calculation requires the most unfavourable combination of lower and/or upper values for independent geotechnical parameters. Appropriate judgement should be applied for selection of representative values of parameters that are interdependent.

Selection of a statistical representative value is typically such that the calculated probability of a worse value governing the occurrence of a limit state is not greater than 5 %. A representative value is, in most cases, a nominal value based on an estimate of a statistical distribution for the representative value of the geotechnical parameter required for a particular calculation model. This estimate is mainly based on a small amount of derived values and general experience. Note that the statistics for derived values are not necessarily representative of in situ conditions. Aleatory uncertainties are covered to some degree. Epistemic uncertainties are not. Furthermore, an estimate of a statistical distribution for a representative value should typically differ from that for a derived value. Where cautious, statistical equivalence is sometimes accepted in practice.

Uncertainty modelling such as described in DNV GL (2017a) will typically require quantification of the epistemic uncertainties of derived values. Such quantification is challenging as it will require large data sets within a 3D context and judgement on selection of adjustment factors.

Hicks (2013) and Baecher and Christian (2003) illustrate that statistical methods for selection of a representative value can be feasible in some situations. Usually, such methods should allow for incorporation of a-priori knowledge of comparable experience with geotechnical parameters, for example by Bayesian methods, as necessary. Variance reduction methods may be applied where appropriate.

In principle, spatial ground variability affects:

- The mean (X_m), standard deviation (SD) and probability density function (pdf) of the ground property for the location under consideration, including any depth trend.
- The scale of fluctuation (θ) of the ground property, which is the distance over which the property values are significantly correlated; the scale of fluctuation in the (near) horizontal plane is often much larger than in the vertical direction, i.e. $\theta_h \gg \theta_v$, for example due to the process of deposition.
- The limit state under consideration, particularly relating to the geometrical quantities of the structure that interacts with the ground, the nature of the applied actions and the volume of ground that represents the domain of influence with respect to the limit state.

The pdf required for the representative value(s) should take account of the spatial variability of ground property values and the limit state under consideration, and thus may differ considerably from the underlying pdf for the location under consideration (Figure 1). If the domain of influence is represented by the dimension D, the representative value will be a function of the ratio θ/D and will generally lie within the following limits:

- For relatively large values of θ/D , there may be considerable uncertainty regarding the property value governing the structure response. Specifically, although the occurrence of

the limit state will generally be governed by the “local” mean, there will be uncertainty about what that mean actually is. The representative value may then be represented by the 5 percentile of the underlying pdf (Figure 1a);

- For intermediate values of θ/D , the representative value may be estimated from a pdf with a reduced variance to account for averaging of properties. However, account should also be taken of any apparent reduction in the property mean due to the tendency for failure to follow the path of least resistance (Figure 1b);
- For small values of θ/D , there is considerable averaging of property values over potential failure surfaces and the response of the structure may be reasonably represented by a cautious estimate of the mean over the failure surface. For the assumption of a normal distribution of X , this is equivalent to a cautious estimate of X_m , the mean of the underlying distribution (Figure 1c).

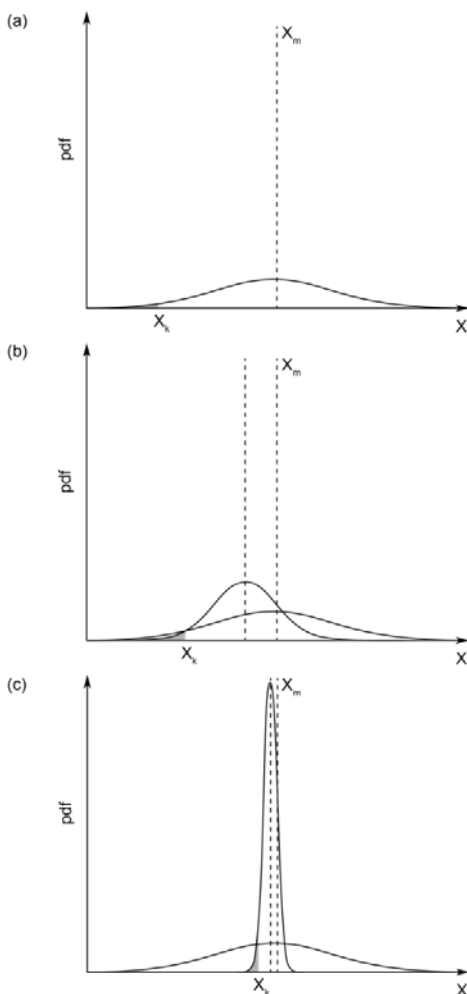


Figure 1: Estimation of representative value and pdf (after Hicks, 2013): (a) X_k based on underlying pdf (for large θ/D); (b) X_k based on modified pdf (for intermediate θ/D); (c) X_k based on modified pdf (for small θ/D)

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