INTERNATIONAL STANDARD

ISO 19901-8

Second edition
2023-09

Oil and gas industries including lower carbon energy — Offshore structures —

Part 8: Marine soil investigations

Industries du pétrole et du gaz y compris les énergies à faible teneur en carbone — Structures en mer —

Partie 8: Investigations des sols en mer

Reference number
ISO 19901-8:2023(E)

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Foreword

ISO (the International Organization for Standardization) is a worldwide federation of national standards bodies (ISO member bodies). The work of preparing International Standards is normally carried out through ISO technical committees. Each member body interested in a subject for which a technical committee has been established has the right to be represented on that committee. International organizations, governmental and non-governmental, in liaison with ISO, also take part in the work. ISO collaborates closely with the International Electrotechnical Commission (IEC) on all matters of electrotechnical standardization.

The procedures used to develop this document and those intended for its further maintenance are described in the ISO/IEC Directives, Part 1. In particular, the different approval criteria needed for the different types of ISO documents should be noted. This document was drafted in accordance with the editorial rules of the ISO/IEC Directives, Part 2 (see www.iso.org/directives).

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For an explanation of the voluntary nature of standards, the meaning of ISO specific terms and expressions related to conformity assessment, as well as information about ISO's adherence to the World Trade Organization (WTO) principles in the Technical Barriers to Trade (TBT), see www.iso.org/iso/foreword.html.

This document was prepared by Technical Committee ISO/TC 67, *Oil and gas industries including lower carbon energy*, Subcommittee SC 7, *Offshore structures*, in collaboration with the European Committee for Standardization (CEN) Technical Committee CEN/TC 12, *Materials, equipment and offshore structures for petroleum, petrochemical and natural gas industries*, in accordance with the Agreement on technical cooperation between ISO and CEN (Vienna Agreement).

This second edition cancels and replaces the first edition (ISO 19901-8:2014), which has been technically revised.

The main changes are as follows:

— application classes for in situ testing tools are removed and replaced by an assessment of documented calibration results and uncertainty analyses;

— new procedures for calibration and verification of cone penetrometers are introduced with reference to the latest edition of ISO 22476-1.

— references to project specifications for technical details have been reduced where possible and roles and responsibilities have been further clarified.

— title and scope change adopted as per Technical Management Board Resolution 53/2022.

A list of all parts in the ISO 19901 series can be found on the ISO website.

Any feedback or questions on this document should be directed to the user's national standards body. A complete listing of these bodies can be found at www.iso.org/members.html.

Introduction

The International Standards on offshore structures prepared by TC 67/SC 7 (ISO 19900, the ISO 19901 series, ISO 19902, ISO 19903, ISO 19904, ISO 19905 series and ISO 19906) constitute a common basis covering those aspects that address design requirements and assessments of all offshore structures used by the petroleum and natural gas industries worldwide. Through their application, the intention is to achieve reliability levels appropriate for manned and unmanned offshore structures, whatever the nature or combination of the materials used. Application specific requirements for different energy industries are referencing relevant overarching standards. For example, for the offshore wind industry the IEC standards IEC 61400-1 and IEC 61400-3-1 outline the normative design requirements (e.g. return periods) for offshore turbine support structures.

Structural integrity is a concept comprising models for describing actions, structural analyses, design rules, safety elements, workmanship, quality control procedures and national requirements, all of which are mutually dependent. The modification of one aspect of design in isolation can disturb the balance of reliability inherent in the overall concept of structural integrity (see ISO 19900). The implications involved in modifications, therefore, should be considered in relation to the overall reliability of all offshore structural systems.

A marine soil investigation is only one of many possible marine site investigations as illustrated in Figure 1. The scope of a marine soil investigation, such as field programme, equipment to be used, laboratory testing programme, soil parameters to be established and reporting, is usually defined in project specifications based on important factors, such as type of structures involved, type of soil conditions expected, regional or site-specific investigation, preliminary or final soil investigations. The reporting can comprise anything from field data only to reporting of soil parameter values.

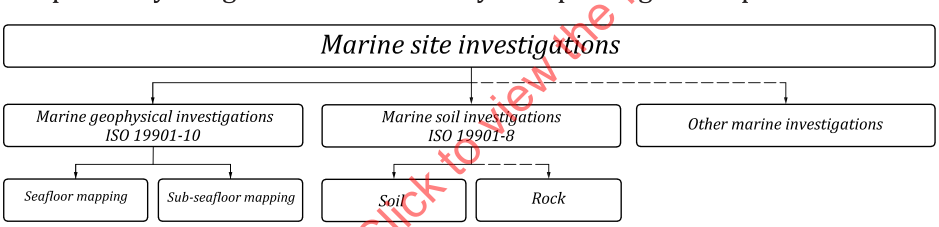


Figure 1 — Marine soil investigations shown as one of many types of marine site investigations.

Use of this document is based on the following assumptions:

— communication takes place between geophysical and geotechnical specialists for defining the scope of the marine soil investigation based on the results of a geophysical investigation (see ISO 19901-10);

— communication takes place between geotechnical personnel involved in marine soil investigations and the personnel responsible for foundation design, for construction and for installation of the offshore structures;

— soil data are collected, documented and interpreted by trained personnel;

— the project-specific scope of work for marine soil investigations is defined by one or more project specifications.

The detailed requirements for equipment and methods given in this document are only applicable if relevant for the scope of work defined in the project specifications.

This document is intended to provide flexibility in the choice of marine soil investigation techniques without hindering innovation.

In this document, the following verbal forms are used:

— "shall" indicates a requirement;

— “should” indicates a recommendation;

— “can” indicates a possibility or a capability;

— “may” indicates a permission.

Information marked as “NOTE” is intended to assist the understanding or use of the document. “Notes to entry” used in Clause 3 provide additional information that supplements the terminological data and can contain requirements relating to the use of a term.

Annex A gives additional information intended to assist the understanding or use of this document. The clause numbers in Annex A correspond to the normative main text to facilitate easy cross-referencing. Annex B covers conduct of laboratory tests as part of marine soil investigations.

Oil and gas industries including lower carbon energy — Offshore structures —

Part 8:
Marine soil investigations

1 Scope

This document specifies requirements and provides recommendations and guidelines for marine soil investigations regarding:

a) objectives, planning and execution of marine soil investigations;

b) deployment of investigation equipment;

c) drilling and logging;

d) in situ testing;

e) sampling;

f) laboratory testing;

g) reporting.

Although this document focuses on investigations of soil, it also provides guidance, with less detail, for investigations of chalk, calcareous soils, cemented soils and weak rock.

Foundation design is not covered by this document.

NOTE 1 ISO 19901-4 and the respective design standards covering foundation design for the specific types of offshore structures to meet the requirements of application specific standards are given on the ISO website.

The results from marine geophysical investigations are, when available and where appropriate, used for planning, optimization and interpretation of marine soil investigations.

This document neither covers the planning, execution and interpretation of marine geophysical investigations nor the planning and scope of geohazard assessment studies, only the corresponding marine soil investigations aspects thereof.

NOTE 2 ISO 19901-10 covers the planning, execution and interpretation of marine geophysical investigations.

This document specifies requirements and provides guidance for obtaining measured values and derived values. This document excludes requirements for determination of design values and representative values. Limited guidance is provided in 11.3 related to data interpretation.

This document is intended for clients, soil investigation contractors, designers, installation contractors, geotechnical laboratories and public and regulatory authorities concerned with marine soil investigations for any type of offshore structures, or geohazard assessment studies.

2 Normative references

The following documents are referred to in the text in such a way that some or all of their content constitutes requirements of this document. For dated references, only the edition cited applies. For undated references, the latest edition of the referenced document (including any amendments) applies.

ISO 14688-1, *Geotechnical investigation and testing — Identification and classification of soil — Part 1: Identification and description*

ISO 14688-2, *Geotechnical investigation and testing — Identification and classification of soil — Part 2: Principles for a classification*

ISO 14689, *Geotechnical investigation and testing — Identification, description and classification of rock*

ISO/IEC 17025, *General requirements for the competence of testing and calibration laboratories*

ISO 19900, *Petroleum and natural gas industries — General requirements for offshore structures*

ISO 22476-1, *Geotechnical investigation and testing — Field testing — Part 1: Electrical cone and piezocone penetration test*

ISO/IEC Guide 99, *International vocabulary of metrology — Basic and general concepts and associated terms (VIM)*

3 Terms and definitions

For the purposes of this document, the terms and definitions given in ISO 14688-1, ISO 14688-2, ISO 14689 and the following apply:

ISO and IEC maintain terminology databases for use in standardization at the following addresses:

— ISO Online browsing platform: available at https://www.iso.org/obp

— IEC Electropedia: available at https://www.electropedia.org/

3.1
accuracy
closeness of agreement between a measured quantity value and a true quantity value of a measurand

[SOURCE: ISO/IEC Guide 99:2007, 2.13]

3.2
borehole geophysical logging
measurement of physical properties of a borehole and/or the surrounding soil, obtained by one or more logging probes deployed in the borehole

3.3
client
party or person with overall responsibility for the marine soil investigation, including preparation of project specifications

3.4
coordinate reference system
coordinate system that is related to an object by a datum

Note 1 to entry: Geodetic and vertical datums are referred to as reference frames.

3.5
contractor
party or person responsible for an assigned scope of work described in project specifications

3.6
derived value
value of a geotechnical parameter obtained from test results by theory, correlation or empiricism

3.7
design value
value derived from the representative value for use in the design verification

[SOURCE: ISO 19900:2019, 3.14]

3.8
disturbed sample
sample whose soil structure, water content and/or constituents have changed as a result of sampling and handling

3.9
drained condition
condition whereby the applied stresses and stress changes are supported entirely by the soil skeleton and do not cause a change in pore pressure

3.10
drilling mud
drilling fluid
fluid pumped down a rotary drilled borehole to facilitate the drilling process

Note 1 to entry: The hardware associated with handling drilling fluids is commonly prefixed 'mud' (e.g. mud tank, mud pump, mud valve). Drilling parameters associated with drilling fluids are similarly prefixed (mud pressure, mud flow, etc.).

3.11
geohazard
geological condition that has the potential to have adverse effects on persons, operations, offshore structures or the environment

3.12
ground model
2- or 3-dimensional representation of the seafloor (bathymetry) and, where applicable, the sub-seafloor conditions, at a given time, that is specific to the offshore structure(s) considered

3.13
ground truthing
integration of seafloor or sub-seafloor geophysical data with data acquired by marine soil investigation and other data

3.14
in-pipe logging
borehole geophysical logging (3.2) in a section of the borehole with drill pipe between the tool and the borehole wall

Note 1 to entry: The number of parameters that can be usefully measured in these circumstances is restricted.

3.15
inclination
the angular deviation of the cone penetrometer from the vertical.

3.16
intact sample
sample that was collected with intention to preserve its in situ characteristics

3.17
integrated geoscience study
combination of geophysical data, a model for geological processes and geotechnical data for development of a *ground model* (3.12)

3.18
marine geophysical investigation
type of marine site investigation of seafloor or sub-seafloor that uses non-destructive methods requiring marine deployment of geophysical tools

Note 1 to entry: See Figure 1.

3.19
marine site investigation
type of investigation at an offshore or nearshore site

EXAMPLE Marine soil investigation, marine geophysical investigation, marine biological investigation, metocean investigation. See Figure 1.

3.20
marine soil investigation
type of *marine site investigation* (3.19) whose primary objective is to obtain reliable and representative soil data for characterization of the seabed soil conditions to facilitate the design of offshore structures and/or for *geohazard* (3.11) evaluation

Note 1 to entry: See Figure 1 and ISO 19901-10.

3.21
measured value
value that is measured in a test

3.22
nominal value
value assigned to a variable specified or determined on a non-statistical basis, typically acquired experience or physical conditions, or as published in a recognized code or standard

3.23
open-hole logging
borehole geophysical logging (3.2) in a section of the borehole without, for example, casing or drill pipe between the tool and the borehole wall

3.24
project specification
scope of work for *marine soil investigations* (3.20) assigned by the *client* (3.3) to a *contractor* (3.5)

3.25
rat hole
additional depth drilled at the end of the borehole (beyond the last zone of interest) to ensure that the zone of interest for *borehole geophysical logging* (3.2) can be fully evaluated

Note 1 to entry: The rat hole allows tools at the top of the logging string to reach and measure the deepest zone of interest.

3.26
reconstituted specimen
laboratory specimen prepared by mixing a soil sample to specified state using a specified procedure

3.27
remoulded sample
remoulded specimen
laboratory specimen that is thoroughly reworked mechanically at a constant water content

3.28
remoulded shear strength
shear strength of remoulded soil

3.29
representative value
value assigned to a basic variable for verification of a limit state in a design/assessment situation

[SOURCE: ISO 19900:2019, 3.40]

3.30
residual shear strength
shear strength at large strains where shear stress versus strain levels off to a constant value

3.31
sample
portion of soil or rock recovered from the seabed by sampling techniques

3.32
sample quality
classification of soil *samples* (3.31) and specimens based on qualitative or quantitative techniques for assessment of the degree of inevitable disturbance induced by the sampling, transportation, handling and storage processes

Note 1 to entry: Sample quality criteria for low to medium OCR clays, where the sample quality is based on measured volume change from laboratory consolidation tests, are given in Table 9.

3.33
seabed
materials below the seafloor

Note 1 to entry: Sub-seafloor can also be used as an equivalent term (See ISO 19901-10).

3.34
seafloor
interface between the sea and the *seabed* (3.33)

3.35
settlement
elastic or permanent downward movement of a structure as a result of its own weight and other actions

3.36
site
defined investigation area, including vertical extent.

3.37
soil parameter
soil property that can be quantified by a descriptor or a value

Note 1 to entry: Cone resistance and undrained shear strength are examples of soil parameters; low strength and very dense are examples of descriptors; nominal value, measured value and derived value are examples of types of values that can be determined for a soil parameter.

3.38
specimen
part of a *sample* (3.31) used for a laboratory test

3.39
strength index test
test that yields an indication of the shear strength

3.40
swelling
expansion due to reduction of effective stress, resulting from either reduction of total stress or absorption of (in general) water at constant total stress

Note 1 to entry: Swelling includes the reverse of both compression and consolidation.

Note 2 to entry: Exsolution of dissolved gas due to stress relief during sampling can cause significant swelling in samples.

3.41
taper angle
rate, in degrees, at which the outside diameter of the sampler cutting shoe reduces

3.42
uncertainty
non-negative parameter that characterizes the dispersion of the quantity values that are being attributed to a measurand

[SOURCE: ISO/IEC Guide 99:2007, 2.26]

3.43
undisturbed sample
sample (3.31) in which no change of practical significance has occurred in the soil characteristics

3.44
undrained condition
condition whereby the applied stresses and stress changes are supported by both the soil skeleton and the pore fluid and do not cause a change in volume

3.45
undrained shear strength
maximum shear stress at yielding or at a specified maximum strain in an *undrained condition* (3.43)

Note 1 to entry: Yielding is the condition of a material in which a large plastic strain occurs at little or no stress increase.

4 Symbols, units and abbreviated terms

4.1 Symbols

A	projected area of the miniature cone, T-bar or ball penetrometer
A_c	nominal cross-sectional area of a cone penetrometer
A_n	area of the load cell or shaft on which pore pressure can act
A_p	projected area of the penetrometer in a plane normal to the shaft
A_s	surface area of the friction sleeve
A_{sb}	cross-sectional area of the bottom of the friction sleeve
A_{sp}	cross-sectional area of the ball or T-bar penetrometer connecting shaft
A_{st}	cross sectional area of the top of the friction sleeve
a	net area ratio of a cone penetrometer, ball or T-bar penetrometer
B	Skempton's pore pressure parameter

B_q	pore pressure ratio
b	net area ratio of the friction sleeve
C_A	area ratio of sampling tube
C_i	inside clearance ratio
C_c	compression index
c_v	coefficient of consolidation
c'	effective stress cohesion intercept
d_w	water depth
D	diameter of miniature penetrometer
D_i	inner diameter of sampling tube
D_o	outer diameter of sampling tube
D_1	diameter of miniature penetrometer in earlier test, for calculation of edge-to-edge test spacing
D_2	diameter of miniature penetrometer in later test, for calculation of edge-to-edge test spacing
e	void ratio
e_{min}	minimum void ratio
e_{max}	maximum void ratio
e_0	initial void ratio
F	measured axial force on the ball or T-bar relative to zero at seafloor
f_s	sleeve friction
F_r	friction ratio (= f_s/q_{net})
F_{thrust}	applied thrust to push rod
G	shear modulus
G_{max}	initial (small strain) shear modulus
G_s	specific gravity of solid particles
g	acceleration due to gravity (= 9,81 m/s^2),
H	specimen height
h_e	height of the cylindrical extension
h_s	height of drill pipe above main deck level
h_d	height of main deck level above the sea level
h_{sf}	height of reference point above seafloor
i	inclination of the cone penetrometer

I_L	liquidity index
I_P	plasticity index
K_0	coefficient of earth pressure at rest (= σ'_{h0} /σ'_{v0})
L	length of T-bar
L_{string}	length of drill string or push rods deployed downhole, in relation to a fixed datum point located either at the seafloor (seafloor-founded systems) or at the drill floor (vessel-drilling systems)
n	number of measurements or observations
p'_f	mean effective stress at s_u
$p_0{}'$	in situ vertical effective stress (=σ'_{v0})
Q	force in the direction of miniature ball or T-bar penetration
Q_t	normalized cone resistance
q_{ball}	net ball resistance
$q_{ball,rem}$	net remoulded ball resistance
q_c	cone resistance
q_{ci}	cone resistance immediately before the penetration interruption in a PPDT
$q_{c;max}$	is the maximum value of cone resistance measured during the penetration phase of the test
q_m	ball or T-bar resistance
q_{MCPT}	penetration/extraction resistance of miniature cone penetrometer
q_{MTPT}	penetration/extraction resistance of miniature T-bar penetrometer
q_{MBPT}	penetration/extraction resistance of miniature ball penetrometer
q_t	cone resistance corrected for pore water pressure effects
q_{net}	net cone resistance (= q_t -σ_{v0})
$q_{T\text{-}bar}$	net penetration resistance for a T-bar penetrometer
$q_{T\text{-}bar,rem}$	net remoulded penetration resistance for a T-bar penetrometer
Ra	surface roughness of a cone penetrometer
R_{cla}	centre-line average roughness of soil-steel interface
R_i	individual roughness value of soil-steel interface
R_f	friction ratio (= f_s/q_c)
S_t	soil sensitivity
$s_u = c_u$	undrained (intact) shear strength of soil
s_{uC}	static triaxial compression undrained shear strength
s_{uD}	static DSS undrained shear strength

s_{uE}	static triaxial extension undrained shear strength
s_{ur}	remoulded undrained strength
t	time
T_0	temperature of the fluid in thermostat bath 2 at 0 °C
T_{30}	temperature of the fluid in thermostat bath 1 at 30 °C
T_a	temperature recorded by a temperature sensor in the cone penetrometer
t_{100}	time to end of primary consolidation at the final consolidation step
U	normalized excess pore pressure
U_c	expanded measurement uncertainty for calibration of cone resistance (denoted as U_{cqc}) and sleeve friction (denoted as U_{cfs})
U_{class}	measurement uncertainty for the determination of cone penetrometer class
$U_{fs:class}$	measurement uncertainty for sleeve friction for the determination of cone penetrometer class
U_{inc}	expanded measurement uncertainty for cone penetrometer inclination
$U_{qc:class}$	measurement uncertainty for cone resistance for the determination of cone penetrometer class
U_u	expanded measurement uncertainty for pore pressure
$U_{u:class}$	measurement uncertainty for pore pressure for the determination of cone penetrometer class
u_{tr}	Standard measurement uncertainty for repeatability of trigger latency
u_0	estimated or measured in situ equilibrium pore pressure relative to seafloor
u_1	pore pressure measured through a filter location on the cone face (location1)
u_2	pore pressure measured through a filter location in the cylindrical extension of the cone(-location2)
u_{2i}	pore pressure immediately before the penetration interruption in a PPDT
u_3	pore pressure measured through a filter location just above the friction sleeve (location 3)
V	total volume of the specimen
V_0	initial volume of specimen
v_p	compression wave velocity
v_s	shear wave velocity
v_{vh}	vertically (v) propagated, horizontally (h) polarized shear wave velocity
v_{1max}	rate of axial displacement of the loading press
w	water (moisture) content
w_L	Liquid limit
w_P	Plastic limit

Z	height above seafloor for drilling mode in situ probe zero reference readings
z	depth below seafloor
$\delta_{ultimate}$	ultimate interface friction angle
ε_a	axial strain
ε_{af}	expected axial strain at failure
ε_{vol}	volumetric strain
γ	unit weight
γ'	submerged unit weight of soil
γ_s	unit weight of solid particles
γ_w	unit weight of distilled water at +4 °C
ρ	bulk (dry or wet) density of soil
ρ_{max}	maximum index dry density
ρ_{min}	minimum index dry density
ρ_s	particle density of soil grains
ν	Poisson's ratio
ξ	material damping ratio
σ	stress
σ_c	uni-axial compressive strength
σ_v	vertical total stress
σ_h	horizontal total stress
σ'_v	vertical effective stress
σ'_h	horizontal effective stress
σ_{v0}	in situ vertical total stress relative to seafloor
σ'_{v0}	in situ vertical effective stress (= p_0')
σ'_{h0}	in situ horizontal effective stress
σ'_p	preconsolidation stress
τ_f	shear stress at failure
ϕ'	effective friction angle
Δa_{fs}	ambient temperature stability for sleeve friction
Δa_{fsc}	ambient temperature stability for corrected sleeve friction
Δa_{qc}	ambient temperature stability for cone resistance

Δa_{qcc} ambient temperature stability for corrected cone resistance

Δa_{u} ambient temperature stability for pore pressure

Δa_{uc} ambient temperature stability for corrected pore pressure

Δb_{fs} bending influence on sleeve friction

Δb_{qc} bending influence on cone resistance

Δb_{u} bending influence on pore pressure

Δe_{0} change in void ratio during reconsolidation

Δf_{gfs} variation in apparent sleeve friction during verification for temperature influence

Δf_{gfsc} variation in temperature corrected apparent sleeve friction during verification for temperature influence

Δf_{gqc} variation in apparent cone resistance during verification for temperature influence

Δf_{gqcc} variation in temperature corrected apparent cone resistance during verification for temperature influence

Δh_{sr} vertical spacing between the SCPT seismic receivers

Δt_{fs} transient temperature stability for sleeve friction

Δt_{fsc} transient temperature stability for corrected sleeve friction

Δt_{qc} transient temperature stability for cone resistance

Δt_{qcc} transient temperature stability for corrected cone resistance

Δt_{u} transient temperature stability for pore pressure

Δt_{uc} transient temperature stability for corrected pore pressure

ΔV change in specimen volume (with reduction in volume being a positive numerical value)

ΔV_{ms} change in volume of the pore pressure measurement system due to pore pressure change

$\Delta \sigma'_{v}$ change in effective vertical stress

4.2 Units

If there are no special historical reasons for deviating from the units listed below, then the International System of units (SI) should be applied:

force	kN
moment	kN·m
density	kg/m^3
unit weight	kN/m^3
stress, pressure, strength and stiffness	kPa
coefficient of permeability	m/s

coefficient of consolidation m^2/s

4.3 Abbreviated terms

ASV	autonomous Surface Vehicles
BHA	bottom hole assembly
CAU	consolidated anisotropic undrained
CCV	consolidated constant volume
CD	consolidated drained
CIU	consolidated isotropic undrained
CPT	cone penetration test
CPTU	cone penetration test with pore pressure measurement
CRS	controlled rate of strain
CT	computed tomography
CU	consolidated undrained
DNV	Det Norske Veritas
DS	direct shear
DSS	direct simple shear
ERP	emergency response plan
FVT	field vane test
GNSS	global navigation satellite system
HAZID	hazard identification
HAZOP	hazard and operability
HR	high resolution
HSE	health, safety and environment
HSEMS	health, safety and environment management system
HVAC	heating, ventilation and air conditioning
ID	identification
IEC	International Electrotechnical Commission
IL	incremental loading
ISM	International Safety Management
ISO	International Organization for Standardization
ISPS	international ship and port facility security

ISRM	International Society for Rock Mechanics
ISSMGE	International Society for Soil Mechanics and Geotechnical Engineering
JSA	job safety analysis
LAT	lowest astronomical tide
MARPOL	international convention for the prevention of pollution from ships
MBPT	miniature ball penetration test
MCPT	miniature cone penetration test
MSCL	multi-sensor core logging
MSL	mean sea level
MTPT	miniature T-bar penetration test
NPR	needle penetration resistance
OC	over-consolidated
OCR	over-consolidation ratio
PEP	project execution plan
PPE	personal protective equipment
PPDT	pore pressure dissipation test
QA	quality assurance
QC	quality control
QP	quality plan
RFID	Radio-frequency identification
ROP	rate of penetration
ROV	remotely operated vehicle
RS	ring shear
SCPT	seismic cone penetration test
SEM	scanning electron microscopy
SHANSEP	stress history and normalized soil engineering parameters
SIMOPS	simultaneous operations
SOLAS	safety of life at sea
SPT	standard penetration test
SUT	Society for Underwater Technology
TBT	technical barriers to trade

TOC	total organic content
UCT	unconfined compression test
UHR	ultra-high resolution
UUHR	ultra-ultra-high resolution
UU	unconsolidated-undrained
UXO	unexploded ordnance
VSP	vertical seismic profiling
WGS	world geodetic system
WTO	World Trade Organization
YSR	yield stress ratio

5 Objectives, planning and requirements

5.1 Objectives

The objectives of marine site investigations are to make data available at the various project phases. In particular, the acquired data are usually required to enable assessment of the site suitability with respect to the offshore structure and the risk tolerance and exposure level determined for an offshore structure. The planning and determination of tools and techniques shall be done after the objectives of a marine site investigation have been determined.

NOTE 1 ISO 19900 outlines general requirements for offshore structures.

NOTE 2 IEC 61400-1 and IEC 61400-3-1 outline design requirements for offshore wind turbines.

NOTE 3 ISO 31000 provides guidance on risk management principles.

The general objectives of a marine soil investigation are to establish the characteristics and mechanical properties of the seabed by acquisition, evaluation and presentation of geotechnical information derived from methods relying on tools penetrating into the seabed.

5.2 Planning

5.2.1 Sequence of activities

Marine site investigations commonly consist of the following activities, often performed in the sequence listed below and illustrated in Figure 2:

a) desk study, including the evaluation of information available in the public domain, available geophysical data, and the results of any previous marine soil investigations in the area;

b) marine geophysical investigations (guidance provided in ISO 19901-10);

c) marine soil investigations, which can comprise one or more phases to suit the project needs;

d) further integrated geoscience study combining the information gathered in the desk study, marine geophysical investigation and marine soil investigation phases.

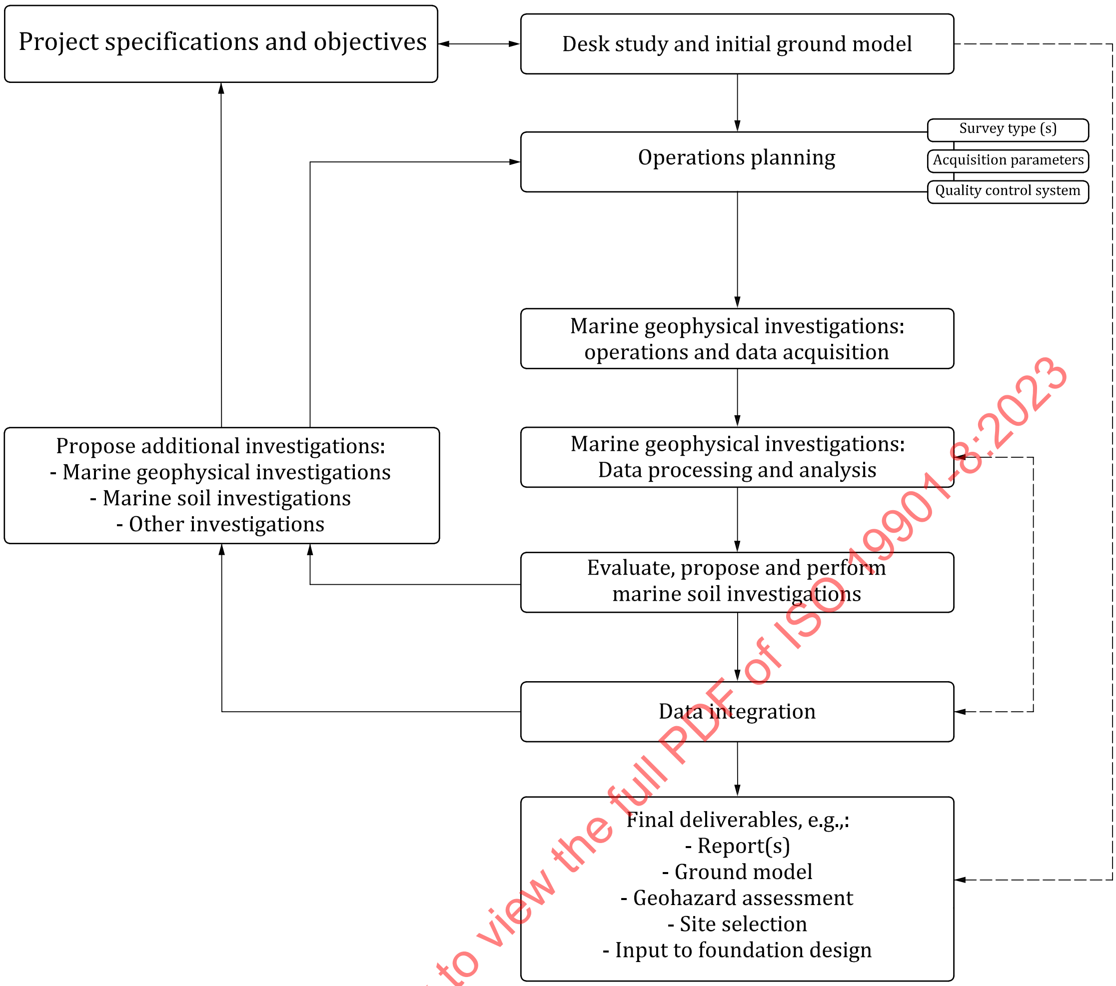


Figure 2 — Typical sequence of activities in marine site investigations

Marine geophysical investigations can comprise:

— bathymetry and seafloor topography, using a multi-beam echosounder;

— seafloor features and obstructions, using methods such as side scan sonar imaging and magnetometer surveys;

— seabed stratigraphy, using methods such as sub-bottom profiling and ultra-high-resolution seismic reflection.

A marine geophysical investigation can cover a large extent of seabed, enabling the identification of localized seabed features, such as the following:

— buried channel, or ice gouge features;

— erosion features;

— slides or shallow mass transport deposits;

— gas, gas pockets or potential gas pockets;

— specific seafloor features (e.g. pockmarks, boulders, concretions, chemo-synthetic communities and seafloor expulsion features, drill cuttings mounds);

— manmade hazards, such as unexploded ordnance (UXO).

A phased approach can be taken to a marine soil investigation. The extent of each phase shall be determined based on the marine site investigation objectives. A 'limited-scope' marine soil investigation is commonly performed in conjunction with marine geophysical investigations for the purpose of preliminary ground truthing. The results of a marine geophysical investigation, alone or in addition to a desk study, are generally not sufficient for detailed design of an offshore structure.

A marine soil investigation can include downhole geophysical logging, in situ testing and sampling operations, with field and onshore laboratory testing performed on any recovered samples, evaluation of geotechnical data and results and reporting. Specific guidance is given on selection of appropriate equipment and procedures in subsequent clauses of this document. The date by which appropriate geotechnical information is required for a given project phase determines when a marine soil investigation should be done. Having the marine soil investigation on the critical path can adversely impact subsequent activities, particularly in case of unexpected findings that require additional work.

Considerations for planning marine site investigations include project phases, requirements of the offshore structure and risk level.

NOTE 1 Examples of project phases are site selection, conceptual design, preliminary design, detailed design, re-certification and decommissioning. Risk level is commonly expressed by probability of failure and consequences of failure. Design codes commonly rely on implicit or explicit risk levels.

NOTE 2 Guidelines concerning the planning and scoping of site investigations for multi-facility sites like offshore wind farms can be found in SUT OSIG (2022)[98].

Geotechnical design requirements for offshore structures shall be established in accordance with ISO 19900 or relevant application specific standards. The geotechnical design requirements depend on several factors that can influence the planning of the scope and extent (spatial area and explored depth) of the marine site investigations, in particular the following:

— project phase, ranging from concept selection to decommissioning;

— type of offshore structure and foundation solutions;

— types and magnitudes of actions (loadings);

— related design situations, such as bearing capacity, stability, settlements/displacements, soil-structure interaction, installation/removal aspects;

— criticality of design situations and the possible need for optimization of design and related geotechnical parameter values;

— methods to be adopted for solving/analysing the various design situations;

— geohazards.

5.2.2 Integrated geoscience studies

It is recommended to perform an integrated geoscience study to facilitate field development, planning and the design of offshore structures.

The principles of a ground model and an integrated geoscience study are given in ISO 19901-10. A multi-disciplinary team with close cooperation of geologists, geophysicists, geotechnical specialists and facility designers, involving both the marine site investigation contractor(s) and the site owner/operator, is essential for an integrated study.

The integrated study starts with an initial ground model based on the desk study. The initial ground model can be updated by a marine geophysical investigation, which is used to interpret general stratigraphy and define the scope of the marine soil investigation. Such an integrated approach allows for a comprehensive definition of an enhanced ground model and identification of potential geohazards in the zone of influence, which is typically the top 100 m to 200 m of soil below the seafloor, but can be 500 m or more in basins with deep soil profiles.

Experienced geotechnical and geophysical practitioners shall be involved in the data integration and development of a ground model. The integration of multiple data sets should consider the following:

a) good communication and interaction in the multi-disciplinary team conducting an integrated study are essential, so that the practical limitations and uncertainties related to each set of marine geophysical and marine soil investigation data or methods are properly understood;

b) the generation of soil parameter values using marine geophysical investigation data is a fairly novel and challenging exercise with inherent uncertainties that should be assessed consistently by integrated teams including geologists, geophysicists, geotechnical specialists and data science specialists.

5.3 Scope of work and development of project specifications

The scope of work and extent of a marine soil investigation shall be given in the project specifications, including any planning activities to be undertaken by the contractor.

The project specifications should state the objectives of the marine soil investigation.

The project specifications should, if relevant, include foundation details of the offshore structures and the selected design approaches.

The project specifications shall assign who is responsible for particular activities (i.e. client or contractor) and shall define the particular activities and corresponding methods to be adopted. Where relevant, the project specifications should refer to methods described in this document. The project specifications can refer to or describe alternative methods not covered by this document. Any references to alternative methods should be accompanied by method-specific information as applicable.

The order of precedence for application of requirements for the execution of work shall be:

1) the project specifications;

2) requirements of this document for the given scope of work;

3) contractor’s normal practice.

NOTE 1 Requirements set by regional, national, and local regulations and standards can be different from those given in this document.

NOTE 2 A project can have more than one project specification.

NOTE 3 The project specifications can be updated during the course of a project when the client and contractor agree to deviate from the original project specifications.

Regarding marine operations, project specifications shall define responsibility for clarifying site clearance issues and any required mitigation measures. Site access or clearance issues typically include:

— navigation requirements such as minimum stand-off distance from existing facilities;

— water depth, seafloor slope and unevenness;

— possibility of encountering shallow gas, UXO and contaminated ground;

— site permits, including protected habitat and sample export.

Where the client holds site access or clearance related information, this shall be made available to the contractor.

The project specifications shall, where applicable, include requirements for:

a) vessel positioning and equipment positioning methods and uncertainties, including the geodetic reference definition parameters and the requirements for reporting these;

b) determination of the extent of borehole geophysical logging, sampling and testing, consisting of the following:

   1) type, number, location and depth of borehole geophysical logging (see Clause 7);

   2) type, number, location and depth of in situ testing locations (see Clause 8);

   3) type, number, distribution, location and depth of sampling locations (see Clause 9);

   4) type and number of offshore and onshore laboratory tests (see Clause 10);

   5) allowable data gaps and any re-testing;

c) sample storage and transportation conditions, both offshore and onshore, including any long-term storage and storage period;

d) choice of appropriate vessel, equipment and techniques (see Clauses 5 to 10);

e) reporting, including presentation of the results of the marine soil investigation, development of a ground model and any associated data interpretation (see Clause 11).

In situ testing data should be reported after the execution of each test, and, where practicable, prior to execution of subsequent tests, for verification purposes. Preliminary results should be reviewed in order to implement any changes to the programme of marine soil investigation before the end of the fieldwork.

Laboratory testing shall be performed in accordance with the project specifications. If not defined in the project specifications, an applicable method in an international standard or technical specification shall be followed. In the absence of an applicable method in an International Standard or Technical Specification, alternative methods, such as those defined in national standards can be followed. Any requirement for an offshore laboratory and offshore laboratory testing shall be included in the project specifications.

It may not be possible to define an item in the project specifications at the outset of a marine soil investigation, such as the consolidation stresses to be used in laboratory tests. In this case, the project specifications should define who is responsible for the activity and the approach to be taken.

5.4 Health, safety and environmental requirements for marine operations

5.4.1 General

The marine soil investigation shall conform to the project HSE requirements given in the project specifications. Marine operations shall be conducted in accordance with a project HSE plan. The project HSE plan shall be prepared before start of the project-specific marine operations and can be part of a PEP. Annex A provides guidance about the HSE plan. A risk assessment for the execution of the marine soil investigation shall be undertaken, and particular requirements/items should be included in the project specification.

5.4.2 Investigation vessel

This subclause applies to seagoing investigation vessels with national registration and an onboard crew. The requirements of this section should also be considered for small boats, jack-up platforms and ASVs, where applicable.

The vessel shall have an HSEMS. The HSEMS should conform to the requirements of the international management codes for safe operation of ships (ISM code) and for pollution prevention (MARPOL code), and with the SOLAS code, or equivalent codes. The vessel should also conform to the ISPS code, where applicable. Guidance on the documentation requirements for the vessel can be found in ISO 19901-6:2009, Table 1 for cargo vessels.

The vessel shall be crewed with a sufficient number of qualified, trained and experienced personnel to perform required marine soil investigation operations, including the operation, maintenance and repair of safety critical equipment.

The safety and well-being of those involved in, and impacted by, the marine soil investigation shall be considered. Personnel involved shall be familiar with safety procedures requisite for the safe completion of tasks in which they participate and general onboard safety, including appropriate offshore survival procedures. The operators of marine soil investigation equipment shall have training and experience in the use of the equipment. Relevant personnel shall be aware of reporting requirements for HSE incidents. HSE incidents or accidents shall be reported.

The investigation vessel shall have appropriate safety equipment to conform to SOLAS codes, including adequate maritime lifesaving equipment, and shall have PPE (e.g. hard hats, safety glasses, flotation work vests, safety lines) as required by individual JSAs and vessel HSEMS, and shall have onboard personnel familiar with and trained in its use.

Special care shall be exercised for 'specially-mobilized' vessels, as opposed to purpose-built vessels, for marine soil investigation, to ensure all temporarily-mobilized equipment is safely installed and operated, and that interfaces between permanent vessel crew, who can be unfamiliar with marine soil investigation operations, and temporary crew members (e.g. positioning crew, client representative(s) and any other specialist crew members) are properly managed.

Marine soil investigation equipment shall not be used before it is safely installed. Lifting equipment, such as cranes, booms, hoists, spreader bars, slings, shall be suitable for the proposed use, checked, inspected or certified, and appropriately tagged or marked. During the marine soil investigation, the vessel should be subject to ongoing safety monitoring.

When operations become unsafe for any reason, such as excessive vessel motion, work shall cease and equipment shall be secured.

5.4.3 Hazardous substances and acoustic noise

Substances, such as drilling mud, laboratory chemicals and radioactive sources, can require special care during handling and storage. The presence and special handling and storage requirements of any hazardous material on board the vessel shall be documented, such as in the project HSE plan.

The project HSE plan should consider the potential impact of in-water investigation equipment on the marine environment (e.g. loss of hydraulic fluid, acoustic noise).

5.4.4 Shallow gas

Shallow gas poses a significant hazard to marine operations. Release of shallow gas from the seabed into the sea and atmosphere also has an environmental impact.

A shallow gas hazard assessment shall be undertaken to identify the likelihood of encountering gas during a marine soil investigation for the specific locations, depths below seafloor and planned activities. The shallow gas hazard assessment should typically consider the following:

a) distance to production infrastructure and historic well-drilling activity;

b) desk study information, including regional geology, possible presence of gas hydrates and relevant offset well information;

c) marine geophysical investigation data, considering type, quality, resolution, depth of penetration below seafloor, and timing of the data acquisition;

NOTE 1 There are several examples where a shallow gas release took place during a marine soil investigation while it was not identified by a site-specific marine geophysical investigation. Reasons can include the presence of dissolved gas in pore water and the occurrence of thin gas bearing layers. Use of geophysical data alone can result in the inevitable level of uncertainty related to the identification of a shallow gas hazard.

d) potential for changes since marine geophysical investigation (e.g. due to well-drilling activity);

e) potential depth, pressure and volume of shallow gas;

NOTE 2 In most cases, no reliable information will be available on potential pressure, volume and formation flow rate of shallow gas.

f) possible sources of shallow gas and likely gas characteristics (composition, toxicity, flammability, density).

A risk assessment shall be undertaken considering the potential consequences of gas release and the mitigating measures available, and documented in the HSE plan.

5.5 Other requirements

5.5.1 Operational requirements

The proposed scope of the marine soil investigation shall be considered when selecting vessels and investigation tools along with the environmental conditions at the planned fieldwork site, such as:

— water depth;

— seafloor slope;

— wind, wave and current conditions;

— possibility of shallow gas/hydrates, shallow water flow and other drilling hazards.

Lifting, handling and deployment facilities (e.g. A-frames, booms, davits, cables, cradles, drilling equipment, winches) for the required logging, sampling and in situ testing equipment, supplies and other equipment shall be suitable for the purposes required and shall be inspected or certified for such use where appropriate. An appropriate inspection regime shall be in place. Dynamic effects from vessel motion and the additional frictional and suction forces imposed on a piece of investigation equipment (e.g. seafloor frame, corer, CPTU) shall be included when determining the maximum anticipated loading on a lifting element, as this additional loading can be significant.

Equipment used during the marine soil investigation should have documentation consisting of operational and maintenance procedures. Where two or more pieces of equipment have a critical interface, those interfaces shall be evaluated to ensure proper operation.

Daily field activities shall be documented during the field operations.

5.5.2 Quality requirements

Geotechnical specialists shall be involved during the planning and preparation of the marine soil investigation project specifications.

The client should have a project quality management system in place, with quality assurance exercised consistently and competently in all phases of investigation and evaluation. The contractor should operate a documented quality management system, such as ISO 9001 or ISO 29001.

A project QP should be incorporated as part of a PEP for the marine soil investigation (see Annex A). The QP should detail the organizational responsibilities, activities, and an index of referenced and applicable procedures to complete the scope of the marine soil investigation.

Quality system audits against the quality management system should be performed. The client and contractor should address and resolve audit reports, recommendations and/or corrective actions within the responsibility of the client and contractor respectively.

5.5.3 Specific considerations for unconventional soils

The requirements and recommendations in this document have been developed primarily for use in conventional soils, i.e. siliceous sands and clays of terrigenous origins which have relatively well understood generic properties, and for which there are well established marine soil investigation practices. When performing a marine soil investigation in frontier areas, or areas known or suspected to contain unconventional soils, this can require special handling or treatment for soils. The equipment, methods and procedures can also require tailoring to investigate properties that can pose particular difficulties for geohazard assessments and engineering. Examples of soils with specific generic properties that can need special attention are listed in Table A.3.

NOTE 1 Calcareous deposits offer an example of how an investigation can be adapted to suit soil type: such soils can be variably cemented and range from lightly cemented with sometimes significant void spaces to extremely well cemented. Therefore, in planning a marine soil investigation, it is important to incorporate sufficient flexibility in the scope of work to switch between soil push sampling, rotary core drilling, and in situ testing techniques as appropriate. In these soils, a carefully developed field and laboratory testing programme can be warranted. ISO 19901-4 provides relevant guidance.

NOTE 2 Relevant properties of frozen soils, permafrost and other geotechnical considerations in arctic environments are discussed in ISO 19906 and ISO 35106.

6 Deployment of investigation equipment

6.1 Non-drilling mode deployment

Non-drilling mode encompasses the practice by which geotechnical testing or sampling tools are initiated at the seafloor, or at the elevation where the equipment is stable, and penetrated to refusal depth or to a predetermined depth outside of a rotary drilled borehole. Tool penetrations generally vary from 0,5 m to greater than 25 m below seafloor, depending on the tool that is deployed and the geology that is encountered. Penetrations in excess of 40 m are possible using specialist equipment.

A vast array of geotechnical testing and sampling equipment is deployed using this technique. The more sophisticated of these tools are landed on the seafloor prior to the commencement of the seabed penetration, i.e. fixed seafloor reference. The simplest of these tools are simply lowered until they encounter the seafloor, allowed to penetrate to refusal under their own weight, then extracted and recovered to the vessel deck, i.e. no fixed seafloor reference.

The quality of data acquired can vary with the level of sophistication of the tools. Sophisticated seafloor-founded, non-drilling mode rigs can acquire high quality samples and/or in situ data. At the other end of the spectrum, non-heave-compensated winch-controlled tools, such as gravity core, Kullenberg piston core and vibrocore samplers, can have a limited capacity to acquire good quality samples.

A common limitation of non-drilling mode equipment relates to the inability to achieve a target penetration depth when a seabed layer is intercepted whose strength exceeds the penetration capability of the tool.

Notwithstanding the above, non-drilling mode tools are widely used for acquiring seabed geotechnical data in favourable geological conditions, due to their generally high level of availability, comparatively lower cost, and their capacity to be deployed from a wide range of vessels.

6.2 Drilling mode deployment

6.2.1 General

In drilling mode, logging, sampling and in situ test tools are deployed into the seabed, initially commencing from the seafloor and thereafter from the bottom of a borehole that is progressed via rotary drilling.

Drilling mode operations can be undertaken from the sea surface (referred to as 'vessel drilling'), from seafloor-founded drill rigs (referred to as 'seafloor drilling') and combinations thereof.

Geotechnical tools are deployed to the bottom of the borehole using a variety of techniques, including wireline, free fall, or via advancement and recovery of the drill string itself. Once located at the bottom of the borehole, the geotechnical tool is penetrated via hydraulic or mechanical means until the maximum stroke length, the borehole target depth, or the mechanical capacity of the system is achieved.

At the completion of a testing or sampling interval, the tool is either recovered to the surface through the drill string (vessel drilling), or temporarily stored on the rig (seafloor drilling). The borehole is then progressed to the next testing or sampling depth.

Further provisions related to rotary drilling techniques are provided in Clause 7.

6.2.2 Vessel drilling

Vessel drilling operations are undertaken from either:

— a spudded, anchored or dynamically positioned floating vessel, such as a drill ship, semi-submersible vessel, barge or similar craft, or

— a stable non-floating platform, such as a jack-up or permanent seabed structure.

Vessel drilling systems are influenced by movements associated with surface environmental conditions and subsea currents. For many vessel drilling systems, the vertical stability of the drill string can be controlled or limited by means of

— a seafloor-founded template with a 'hard-tie system' heave compensator (see Zuidberg et al. (1986) [173]), or

— a heave compensator on the drilling vessel in conjunction with soil resistance that reduces movement of the drill string.

Less sophisticated vessel drilling systems are uncompensated against vessel heave. Further discussion of the effects of vessel heave on drill string stability, and the uncertainty of vertical depth measurements, is provided in A.6.3.

6.2.3 Seafloor drilling

Seafloor drilling systems are landed on the seafloor and all operations are undertaken via remote control with the rig being isolated from the vessel movements and environmental conditions at the sea surface.

The stability of seafloor drilling systems is largely dependent on the capabilities of the footings used to support the rig. Sophisticated footing systems can incorporate the capacity to land on steep slopes, self-level the rig, and monitor the vertical height of the rig relative to the seafloor to enhance the certainty of vertical depth measurements.

6.3 Uncertainty of vertical depth measurements

6.3.1 General

Appropriate measurement of vertical depth assists in ensuring that:

— the vertical positions of data points can be referenced to a vertical datum such as seafloor, LAT or MSL;

— variations in seabed stratigraphy, including seafloor, are adequately defined;

— appropriate overburden and pore water pressure corrections are applied to in situ test data;

— adjacent samples and in situ test data can be precisely correlated;

— appropriate confining stresses are specified during subsequent laboratory test programmes.

6.3.2 Factors affecting the uncertainty of vertical depth measurements

A data point can be defined at any depth below seafloor at which geotechnical data are acquired. The depth uncertainty of a data point can vary depending on:

— water depth;

— soil conditions near seafloor;

— metocean conditions at the time of investigation;

— selected investigation tools;

— the mode by which equipment is deployed;

— the vertical stability of the equipment during the data acquisition phase;

— the uncertainties of the measurement system;

— the uncertainties of estimation of a sample data point within a sampling tool.

The achievable depth uncertainty class when using a vessel-mounted drilling system can depend on the prevailing environmental conditions at the sea surface, the associated vessel motions, the water depth, the capability of a vessel's heave-compensation system, and whether the drill string can be stabilized at the seafloor during downhole data acquisition. In contrast, the depth uncertainty class achieved with a seafloor-founded system can depend on the capabilities of the monitoring system used to estimate the position and levelness of the rig relative to the seafloor, and the length of the drill string or push rods deployed downhole.

Data point depth uncertainty can additionally vary as a result of the performance of a sampling tool.

NOTE The allocation of sample loss within a sampling run is often solely based on engineering judgement. This can introduce additional uncertainty as to the actual depth from which the sample originated.

6.3.3 Depth uncertainty classes

The project specifications should specify the depth uncertainty class, as defined in Table 1, to be achieved in a marine soil investigation based on the intended use of the geotechnical data, with due consideration to the investigation scope, the capabilities of the equipment under consideration and procedures. Multiple depth uncertainty classes can be specified, for example differentiating between data points acquired by in situ testing and sampling operations. The depth uncertainty for a sample data point depends on the uncertainty of the sample position within a sampling tool, i.e. allocation of any sample loss/extension within a sampling run. Guidance is provided in 9.5 and Annex A.

If no depth uncertainty classes are given in the project specifications, a default value Z2 shall apply to data points acquired by borehole geophysical logging and in situ testing. A default depth uncertainty class Z3 shall apply for data points acquired by sampling, except for samplers with no fixed seafloor reference, where a default depth uncertainty class Z5 shall apply (see 9.4 and A.6.3.3).

Table 1 — Depth uncertainty classes for data points relative to seafloor

Depth uncertainty class	Maximum data point depth uncertainty m
Z1	0,1
Z2	0,5
Z3	1,0
Z4	2,0
Z5	>2,0

The depth uncertainty according to Table 1 shall be estimated as combined standard uncertainty in accordance with ISO/IEC Guide 99. Demonstration of the depth uncertainty that can be achieved with proposed investigation equipment shall be documented by the contractor. For cases where depth uncertainty is expected to vary with changes in water depth and/or depth of data point below the seafloor, the variation in depth uncertainty class should be estimated for each intended investigation area. The achieved depth uncertainty class or depth uncertainty classes shall be reported. The reporting on depth uncertainty class shall include the method and numerical values used for estimation of data point uncertainty. The vertical datum should be seafloor as found immediately before the marine operations at the target location.

6.4 Horizontal positioning

The accuracy to which boreholes and/or equipment are located on the seafloor is an important aspect of a geotechnical data set, because it is imperative that:

— current investigation locations are aligned with previous and future marine soil investigations, and the installation of offshore structures;

— known seafloor hazards, including installed offshore structures, natural seabed obstructions, excessive gradients, identified areas of shallow gas, UXO and other features, are avoided with confidence.

The coordinate reference system and corrections to be applied to the data set shall be included in the project specifications.

Specification of horizontal positioning tolerances can be influenced by:

— the type of offshores structures to be installed on the seafloor;

— the type, levels of equipment redundancy and uncertainties of the positioning measuring system;

— the proximity of existing offshore structures, including the estimated uncertainty of their reported positions;

— the known seafloor topography: water depth, seafloor slopes, natural features.

Further guidance on horizontal positioning requirements is provided in ISO 19901-10.

6.5 Interaction of investigation equipment with the upper seabed

The action of landing and operation of equipment on the seafloor can disturb the upper seabed from the applied surcharge pressures or lead to the seafloor template penetrating into the upper seabed. This can be detrimental to data quality, particularly for very soft seabed. Where appropriate, consideration should be given to deployment of specific investigation equipment for acquisition of high-quality data for the upper seabed (e.g. equipment with no fixed seafloor reference, equipment with low bearing pressure, and sophisticated seafloor drills with large, extended footings). Detailed bathymetry information is normally required to confirm that the investigation equipment can be safely operated in order to optimize landing speed and upper soil disturbance during landing.

Reporting shall include an assessment of the interaction of the deployed investigation equipment with the upper seabed, and whether this can have adversely affected the acquired data, including any elevation changes of the investigation equipment and any correction applied to the acquired data. Further guidance is provided in A.6.5.1.

7 Drilling and logging

7.1 General

Drilling operations can be undertaken by various modes of equipment deployment defined within Clause 6.

The delivery of the drilling operation can be fundamental to the overall success of the marine soil investigation.

A borehole is advanced by the combined action of a rotating cutting surface (the drill bit) on material at the bottom of the hole and the flow of drilling fluids that flushes drill cuttings in suspension from the bottom of the hole up the enclosing annulus between the drill pipe and borehole wall (or casing inner wall).

Some soil disturbance ahead of the drill bit is inevitable and can have a measurable effect on data quality. Excessive, or varying, weight on the drill bit tends to increase soil disturbance as the bit impacts undisturbed material at the drill face; rotation speed and torque can also influence sample quality. Excessive drilling fluid pressure can induce hydraulic fracture or erosion of undisturbed material, and excessive drilling fluid flow can soften or erode undisturbed material. Soil disturbance can often be assessed by inspection of the results from the sampling and in situ testing and operational methods can be adjusted as required. For heave compensated drilling the depth of soil disturbance below the drill face, and the magnitude of its effect, tend to be greater for softer soils as many systems require a reaction force provided by the soil to compensate against. For marine drilling operations, the vertical stability of the drill string during borehole advancement is critical to the recovery of high-quality geotechnical data. Fluctuations in applied bit weight, and associated displacements, during sampling and in situ testing operations can be detrimental to data quality. Drill string stability and out-of-straightness is largely affected by the type of equipment used to deploy the drill string. Further information on this issue is provided in Clause 6.

7.2 Project-specific drilling requirements

The choice of drilling equipment suitable for a particular marine soil investigation is generally dictated by a number of key project-specific drilling requirements, which can consist of the following:

- expected soil conditions and possible drilling hazards, such as shallow gas, flowing sands, swelling clays, hydraulic fracture potential;
- required borehole depth and purpose of geotechnical borehole(s):
  - borehole suitable for deployment of specific testing or sampling tools, and/or
  - performance of a pilot hole or borehole to acquire drilling parameters (i.e. destructive drilling);
- site environment and seafloor characteristics (e.g. water depth, expected weather and sea conditions (current, tide, extremes of temperature), proximity of surface, submerged, seafloor or buried infrastructure, seafloor topography, natural seafloor features);
- required depth uncertainty class (see 6.3);
- HSE considerations (e.g. personnel interventions on the drill floor, environmental discharge regulations for drill cuttings and/or mud type);
- Logistical considerations (e.g. project schedule, availability of equipment).

7.3 Drilling objectives and selection of drilling equipment and procedures

Drilling objectives of a marine soil investigation are typically one or more of the following:

— to recover data of sufficient quality, whilst minimising soil disturbance from drilling, where the marine soil investigation results will be used to determine geotechnical parameter values for design purposes;

— to maximize the continuity of data recovered within a borehole (at the possible expense of reduced data quality), for example for geological investigations for which key horizons can be very thin and therefore require a data record as continuous as possible;

— to maximize the speed of drilling, at the expense of data quality and/or continuity, in the interests of providing as much spatial or depth coverage of a site as possible within a given time or budget constraint, for example for hard-layer detection;

The selection of drilling equipment and procedures shall be based on the drilling objectives given in the project specifications, with due consideration to the range of expected soil conditions and drilling hazards that can be encountered. For sampling by rotary core drilling, the drilling system and the associated sampling techniques can conform to the requirements of ISO 22475-1 (mainly for onshore and nearshore soil investigations).

Further guidance on the selection of drilling equipment and procedures is provided in A.7.3.

7.4 Drilling operations plan

A drilling operations plan should be in place before start of drilling activities on site. The purpose of the drilling operations plan is twofold:

1) during the pre-fieldwork phase, to provide assurance that:

   — required information is documented as detailed below;

   — the proposed solution is capable of meeting the objectives of drilling and logging;

   — consideration has been paid to potential drilling hazards (see 7.2);

   — contingency plans are in place in the event of foreseeable drilling-related problems;

2) during the fieldwork phase, to control the execution of the drilling and borehole works.

The level of detail required depends on the complexity of the marine soil investigation, on the tools and techniques to be employed, and on whether any new or modified equipment or procedures are required.

The plan should not hinder the continuous assessment of data quality and if necessary, the adjustment in operational procedures if required during the completion of the work. For the range of possible soil conditions, the drilling operations plan shall document the following components, where applicable:

a) drilling equipment and operational details (see A.7);

b) borehole construction plan, describing the sequence of borehole construction (with drill string details and drill bit selection), identification of possible drilling problems, borehole abandonment procedures and grouting procedures, and contingencies or remediation options (e.g. in the event of stuck string or tool lost in the borehole);

c) mud plan, with the drilling fluid solution adopted and criteria for change of fluids considering potential for adverse chemical reaction (see A.7);

d) requirements for documentation and reporting of drilling parameters;

e) schedule for borehole geophysical logging, in situ testing and sampling;

f) shallow gas plan if potential for a shallow gas release exists (see 5.4.4);

g) grouting plan, with the grouting fluid/additives selected, grouting procedures and requirements for grout testing (i.e. grout cube crush tests and acceptance criteria).

7.5 Recording of drilling parameters

During the process of borehole drilling, the system operational and performance parameters shall be documented. Recording can be done manually (usually by the driller) or by using an automated logging system. When depth resolution of logged parameters better than 0,5 m is required, use of automated logging is the preferred method.

The recorded parameters and information shall include the following:

a) geographical location and borehole name;

b) water depth (e.g. relative to LAT or MSL);

c) type of drilling fluid and properties;

d) type of drill bit and bottom hole assembly;

e) Date and time (dd/mm/yyyy hh:min);

f) penetration (depth to bottom of borehole) (m);

g) ROP (m/min);

h) time to drill each interval (min);

i) duration and reason for any stoppage (min);

j) drilling fluid pressure at the output of the pump (kPa), where available;

k) drilling fluid rate (input) (l/min), where available;

l) drilling fluid recovery rate (l/min), only for riser drilling with recirculation of drilling fluid, and where available;

m) drill head rotational torque (N·m), where available;

n) drill head rotational speed (revolutions per min), where available;

o) weight on bit (kN), where available.

The recording of drilling parameters can provide a valuable data set for use in interpolation/extrapolation of soil data and detection of layering.

NOTE An example of such application for detection of hard layers in carbonate material is given in Becue et al. (1988)[102].

7.6 Borehole geophysical logging

7.6.1 General

Borehole geophysical logging provides enhanced geological and engineering detail over the logged borehole section. Further guidance is provided in A.7.6.

The project specifications should specify the following, where applicable:

a) required measurement types and corresponding logging tools;

b) applicability of logging tool for expected setting, including metocean conditions, depth below seafloor and soil conditions;

c) deployment method (e.g. wireline, open- or cased-borehole logging);

d) minimum number of measurements per logged depth section;

e) required sequence, direction and speed of lifting (or lowering) of the tool for logging;

f) uncertainty of depth below seafloor;

g) required correlations between measurements and geotechnical parameters;

h) number of tools or length of logging section per run;

i) whether a heave-compensated logging line is required;

j) length of rat hole to be drilled.

7.6.2 Reporting of results

Reporting of borehole geophysical logging results shall include, where applicable:

a) geographical location and borehole name;

b) coordinates, including coordinate reference system;

c) water depth (e.g. relative to LAT or MSL);

d) measurement results and derived values of geotechnical parameters;

e) characteristics of the borehole geophysical logging system, including deployment method;

f) details of borehole conditions in accordance with Clause 7;

g) statement on corrections applied to the data;

h) details of the processing performed on the data;

i) distinction on open-hole logging or in-pipe logging;

j) sequence, direction and rate of travel of the logging probes in the borehole;

k) achieved depth uncertainty class according to 6.3.3.;

l) definitions, formulae, assumptions, and limitations of derived values of geotechnical parameters.

8 In situ testing

8.1 General

In situ tools can be deployed via two alternative methods:

1) non-drilling mode, in which tool penetration is initiated at the seafloor and penetrated to refusal, or to a predetermined depth; or

2) drilling mode, where the tool is lowered down a predrilled borehole and in situ testing is initiated at the bottom of the borehole.

The recommendations and requirements given in Clause 6 should be taken into account when planning and performing in situ tests. For tests undertaken in drilling mode, Clause 7 should also be consulted.

Calibration of in situ probe sensors shall be performed in a calibration laboratory that meets the requirements of ISO/IEC 17025. The calibration laboratory should be accredited for the calibration and verification activities required for in situ tools.

In situ tools and probes shall be manufactured using corrosion resistant material applicable to marine conditions.

8.2 General requirements for the reporting of in situ tests

For each test, the following information shall be reported in addition to the test results:

a) site geographical details, including:

  — geographical location;

  — borehole or test title;

  — coordinates, including coordinate reference system given in the project specifications (e.g. WGS 84);

  — water depth (e.g. relative to LAT or MSL);

b) equipment details (as applicable):

  — deployment mode;

  — cone penetrometer class according to Table 2;

  — measuring intervals for calibration of the sensors for test measurements;

c) test details (as applicable):

  — test type;

  — time and date;

  — test/stroke number;

  — current borehole depth;

  — tool type (geometry/capacity);

  — unique equipment/tool number;

  — applied thrust to push rod (F_{thrust}, in kN) during penetration into the seabed;

  — summary of procedures for verification of applied thrust and for verification of achieved penetration;

  — stop criteria/reason for termination;

  — test category in accordance with Table 4.

In addition, the following information should be documented:

d) additional site geographical details, including facility name;

e) additional equipment details (as applicable):

  — equipment name;

  — type of penetration system;

  — borehole progression method;

Any deviations from the test requirements (e.g. testing procedures, tool wear and tool damage) shall be identified and reported.

8.3 Cone penetration test

8.3.1 General

This subclause covers cone penetration tests performed with a CPT and with a CPTU.

8.3.2 Equipment

A standard cone penetrometer is defined as having a nominal cross-sectional area, A_c, of 1 000 mm^2. Cone penetrometers with diameter between 25 mm (A_c = 500 mm^2) and 50 mm (A_c = 2 000 mm^2) may be used for special purposes without the application of correction factors. Cone penetrometers with diameter less than 25 mm or diameter exceeding 50 mm may be used for special purposes, and for such cone penetrometers, suitable correction factors shall be applied, and the values of the correction factors shall be reported.

The geometry, surface roughness and surface hardness of a cone penetrometer shall comply with ISO 22476-1, subject to the following clarifications and amendments:

1) For cone penetrometers with a diameter other than a nominal diameter of 35,7 mm, the dimensional tolerances given for standard A_c = 1 000 mm^2 cone penetrometer should be linearly scaled in proportion to the diameter;

2) The surface area of the friction sleeve, A_s, may differ from 15 000 mm^2 for cone penetrometers with a diameter other than a nominal diameter of 35,7 mm. The nominal ratio of length to diameter of the friction sleeve should be between 3 and 5 (preferably 3,75);

3) For measurement of u_2, the filter element shall be in the cylindrical part of the cone. The cylindrical part of the cone excludes the gap between the cone and the friction sleeve;

4) For measurement of u_2, the diameter of the filter at the start of the test shall be equal to the diameter of the cylindrical part of the cone, applying dimensional tolerance limits in accordance with ISO 22476-1;

5) The diameter of the filter element may differ from the diameter of the friction sleeve.

NOTE 1 The amendments and clarifications above are based on ISO 22476-1.

For pore pressure measurements, the filter element location in the cylindrical extension part of the cone is recommended. Pore pressure measured at this location is denoted u_2 (Location 2). Pore pressures can also be measured on the cone face (u_1) (Location 1) and/or just above the friction sleeve (u_3) (Location 3).

NOTE 2 Pore pressure measurement at Location 1 (u_1) can show lower susceptibility to loss of saturation upon penetration in dense dilating sands, compared to measurements at Location 2 (u_2).

A cone penetrometer designed solely for the measurement of cone resistance may be used, if the following conditions are met:

a) geometry and surface roughness of the cone penetrometer comply with ISO 22476-1, where applicable;

b) downhole deployment in drilling mode; and

c) ground conditions are such that cone resistance values exceeding 120 MPa can be expected.

NOTE 3 The permission for a cone penetrometer with a limited number of sensors allows the use of more robust cone penetrometers. The primary benefit would be increased CPT coverage in hard ground, i.e. an increase in safe deployment limits.

The cone penetrometer and adjoining rod(s) should have the same diameter for at least 400 mm behind the tip for standard A_c = 1 000 mm^2 cone penetrometers. For other size cones, this distance should be linearly scaled in proportion to the diameter.

NOTE 4 Changing rod diameter at a distance of less than 400 mm (for standard A_c = 1 000 mm^2 cone penetrometers) can influence the directional stability of the system and possibly the cone resistance.

8.3.3 Test procedures

8.3.3.1 Selection of equipment and procedures

Equipment and procedures to be used should be selected considering the expected ground conditions and the project specifications.

Cone penetration test results shall be classified considering the cone penetrometer uncertainty characteristics and diagnostic checks of the sensors during the field work, using:

a) cone penetrometer class described in 8.3.3.3;

b) test category described in 8.3.4.

A.8 provides guidance for selection of cone penetrometer class and provides an indication of test categories that can be achieved.

8.3.3.2 Calibration and verification requirements

Cone penetrometers shall be calibrated for the following measurands, where applicable:

a) cone resistance;

b) sleeve friction;

c) pore pressure;

d) area ratios of the cone (*a*) and of the friction sleeve (*b*);

e) inclination.

Calibration shall include determination of dimensions of the cone, friction sleeve and, where applicable, filter element and gaps. Cone penetrometers shall be verified for temperature influence and for bending influence.

Calibration and verification of cone penetrometers shall be in according to ISO 22476-1, subject to the following clarifications and amendments:

1) The calibration interval shall be limited to the following, whichever is earlier:

   — twelve months after first use of a cone penetrometer for a cone penetration test, and

   — up to the date and time when one or of the sensors is overloaded or showing signs of malfunction.

   The period prior to first use shall not exceed 12 months;

2) The statement of conformity with at least one of the cone penetrometer classes shall be according to requirements for the cone penetrometer classes given in 8.3.3.3 of this document;

3) The uncertainty contributions for calibration of the pore pressure sensor may exclude uncertainty of the dimensions of the cone penetrometer;

4) If the pressure vessel for calibration of the pore pressure sensor is filled with gas, then the temperature in the pressure vessel shall be recorded and presented as described for a pressure vessel filled with fluid;

5) For determination of cone and friction sleeve dimensions, the expanded measurement uncertainty for the measuring device for dimensional measurement shall be determined for a coverage factor of $k = 2$;

6) The maximum value of expanded measurement uncertainty for cone penetrometer inclination shall be used for conformity assessment according to 8.3.3.3 of this document;

7) For verification of a cone penetrometer for temperature influence, the apparent temperature T_a refers to the temperature recorded by a temperature sensor in the cone penetrometer that is used for determination of Δa_{qcc}, Δa_{fsc} , Δa_{uc} , Δt_{qcc}, Δt_{fsc} or Δt_{uc}. The temperatures T_0 and T_{30} refer to the temperatures of the fluid in respectively thermostat bath 2 at 0 °C and thermostat bath 1 at 30 °C;

8) For verification of a cone penetrometer for temperature influence, the position requirements (e.g. fluid level) for the cone penetrometer in thermostat bath 1 shall be as described for thermostat bath 2;

9) For verification of a cone penetrometer for temperature influence, test records and test report may be split, similar to the test records and calibration report for the pore pressure sensor;

10) For verification of a cone penetrometer for temperature influence, the test report shall include, where applicable, variation in cone resistance (Δf_{gqc} and Δf_{gqcc}) and variation in sleeve friction (Δf_{gfs} and Δf_{gfsc}) determined as defined by ISO 22476-1;

11) For verification of a cone penetrometer for bending influence, clamping of the cone penetrometer shall not be done with the loading device. The loading device shall be independent from the frame for clamping;

12) For cone penetrometer classes 1 and 2 according to 8.3.3.3 of this document, the verification of a cone penetrometer for temperature influence and for bending influence may be limited to reporting the results for 20 or more tests on cone penetrometers of the same design, the same type and made according to the same manufacturing and assembly process. The verification results for these penetrometers shall all meet the decision rules for respectively temperature influence and bending influence defined in Table 2 of this document.

NOTE The amendments and clarifications above are based on ISO 22476-1.

Test records, calibration reports and test reports for calibration and verification shall include:

i. description and identification of the cone penetrometer;

ii. identification of the calibration laboratory (or calibration laboratories);

iii. description of the calibration laboratory environment during calibration, average temperature and range of variation;

iv. identification and uncertainty characteristics of reference instruments used for calibration and verification;

v. details of any adjustment or repair made prior to calibration;

vi. date of calibration, measuring intervals for calibration and calibration results;

vii. analysis of the uncertainty of the calibration for each of the penetrometer sensors;

viii. statement of conformity to cone penetrometer class according to Table 2;

ix. statement identifying how the measurements are metrologically traceable.

8.3.3.3 Cone penetrometer class conformity assessment

Cone penetrometers shall be classified following the scheme presented in Table 2. Required values of the parameters of Table 2 ($U_{qc:class}$, $U_{fs:class}$, $U_{u:class}$, Δa_{qc}, Δa_{fs}, Δa_{u}, Δt_{qc}, Δt_{fs}, Δt_{u}, Δa_{qcc}, Δa_{fsc}, Δa_{uc}, Δt_{qcc}, Δt_{fsc}, Δt_{uc}, Δb_{qc}, Δb_{fs} and Δb_{u}) shall be calculated in accordance with 8.3.3.2, where applicable.

For the fulfilment of cone penetrometer classes 1+ and 1, the cone penetrometer shall have cone resistance, sleeve friction and pore pressure sensors. The cone penetrometer shall meet all the requirements for each of the sensors for a given cone penetrometer class to be achieved.

Conformity to a cone penetrometer class shall be determined after each calibration of the cone penetrometer.

Table 2 — Classification of cone penetrometers under laboratory conditions

Cone penetrometer class	Measurand	Allowable maximum measurement uncertainty [a]	Ambient temperature stability [c]	Transient temperature stability [c]	Bending influence [c]
		$U_{qc:class}$ $U_{fs:class}$ $U_{u:class}$	Δa_{qc} or Δa_{qcc} $\Delta a_{fs,}$ or Δa_{fsc} Δa_{u} or Δa_{uc}	Δt_{qc} or Δt_{qcc} Δt_{fs} or Δt_{fsc} Δt or Δt_{uc}	Δb_{qc} Δb_{fs} Δb_{u}
1+	Cone resistance	35 kPa or 1 %	1 kPa/°C	5 kPa/°C	0,3 kPa/N
	Sleeve friction	5 kPa or 1 %	0,1 kPa/°C	0,5 kPa/°C	0,1 kPa/N
	Pore pressure	25 kPa or 0,5 %	0,1 kPa/°C	0,5 kPa/°C	0,05 kPa/N
1	Cone resistance	35 kPa or 1 %	2 kPa/°C	10 kPa/°C	0,3 kPa/N
	Sleeve friction	5 kPa or 1 %	0,1 kPa/°C	0,5 kPa/°C	0,1 kPa/N
	Pore pressure	25 kPa or 0,5 %	0,1 kPa/°C	0,5 kPa/°C	0,05 kPa/N
2	Cone resistance	100 kPa or 2 %	10 kPa/°C	50 kPa/°C	1 kPa/N
	Sleeve friction	15 kPa or 2 %	0,5 kPa/°C	2,5 kPa/°C	0,5 kPa/N
	Pore pressure [b]	50 kPa or 1 %	1 kPa/°C	2,5 kPa/°C	0,1 kPa/N

[a] The allowable maximum measurement uncertainty is the larger value of the two quoted. The relative uncertainty applies to the measured value and not the measuring range.

[b] Pore pressure requirements apply only to CPTU.

[c] The values of ambient temperature stability, transient temperature stability and bending influence represent the maximum allowable for each cone penetrometer class.

Conformity to a cone penetrometer class shall apply when all the following requirements are met, where applicable:

a) each of the parameters given in Table 2 meets the requirements of Table 2 for each of the measurands for a given cone penetrometer class;

b) resolution and output stability, as defined in 8.3.4, are better than one-third of the allowable maximum measurement uncertainty values of Table 2, for a given cone penetrometer class;

c) each of the expanded measurement uncertainty values for cone penetrometer inclination is better than 1 degree, for any of the cone penetrometer classes.

The determination of cone penetrometer class includes a smaller set of uncertainty contributions (Table 3), $U_{qc:class}$, $U_{fs:class}$ and $U_{u:class}$ as defined in ISO 22476-1, than those required for expanded measurement uncertainty for the calibration, U_{cqc}, U_{cfs} and U_{u} as defined in ISO 22476-1. A cone penetrometer can thus conform to a cone penetrometer class, where uncertainties presented in a calibration test report are higher than values of Table 2.

A cone penetrometer can conform to more than one cone penetrometer class for the case of multiple intervals for calibration.

For cone penetrometers that rely on real-time or post-processing correction for temperature influence on CPT results, the corrected values of ambient temperature stability (Δa_{qcc}, Δa_{fsc} and Δa_{uc}) and the corrected values of transient temperature stability (Δt_{qcc} , Δt_{fsc} , Δt_{uc}), as defined in ISO 22476-1, can be used for the assessment of cone penetrometer class in addition to the assessment without applying corrections. The equations applied for correction shall be reported.

Table 3 — Uncertainty contributions for cone penetrometer measurands

Uncertainty contribution[a]	Measurand		
	Cone resistance	**Sleeve friction**	**Pore water pressure**
Reproducibility	√	√	
Repeatability	√	√	√
Resolution	√	√	√
Zero drift	√	√	√
Interpolation	√	√	√
Reversibility	√	√	√
Apparent load transfer	√	√	

[a] Definitions and formulae for the calculation of uncertainty contributions listed are presented in ISO 22476-1.

8.3.4 Procedures for testing offshore

A filter element can be used for several tests if the pore pressure response is closely monitored for each test and the filter element is replaced when an unsatisfactory response is observed.

Reference readings and values of output stability for cone resistance, sleeve friction and pore pressure, where applicable shall be reported before and after the penetration phase of the test. For reference readings, the output of cone resistance, sleeve friction and, if applicable, the pore pressure of the cone penetrometer shall be continuously recorded for a period of one minute at a frequency of at least 1 Hz before and after the penetration phase of the test under conditions of no soil pressure and under similar temperature and ambient pressure conditions. For each of these periods, reference readings for cone resistance, sleeve friction and pore pressure shall be calculated as the mean of the recorded output values. Output stability is defined as the subtraction of minimum output value from maximum output value, for each period of 1 min.

NOTE 1 Temperature changes in the cone penetrometer can cause significant sensor drifts, so it is important to ensure similar temperatures for the cone penetrometer while taking reference readings before and after the test.

NOTE 2 Soil can adhere to the cone penetrometer or get trapped in the gaps below and above the friction sleeve, which can influence the reference readings. The cone penetrometer can be cleaned or rinsed before taking reference readings.

For seafloor-based drilling and non-drilling systems, reference readings can be made at deck level, at the seafloor or a short, fixed distance above. For CPT/CPTU during vessel drilling operations, the reference readings shall be at deck level. The output of cone resistance, sleeve friction and, if applicable, the pore pressure of the cone penetrometer at the bottom of the borehole should be compared to theoretically calculated values.

For quality control and early diagnosis of malfunction, additional checks of output stability can be taken at seafloor level for seafloor CPT/CPTU and at the bottom of the borehole for drilling mode (down-hole mode) CPT/CPTU. The penetration phase shall not commence until the observed output drift is less than 3 kPa per minute for cone resistance, less than 0,15 kPa per minute for sleeve friction, and less than 0,5 kPa per minute for pore pressure. Output drift is defined as the difference between the output at the start and end of a one minute period. Output drift can be calculated by applying a moving average to sensor output.

The rate of penetration derived from the depth sensor shall be (20 ± 5) mm/s. In rock and soils such as cemented soils, dense sands and gravels, penetration rates less than (20 ± 5) mm/s may be used, to prevent damage to the cone penetrometers and push rods or to achieve larger depths. The rate of penetration may be less than (20 ± 5) mm/s immediately before and immediately after a penetration interruption, including specific stationary testing (e.g. PPDT, SCPT), start and end of penetration and add-on of a push rod. These variations on the rate of penetration shall be reported.

The length of each stroke should be as long as possible, with due consideration for project specifications as well as the mechanical and strength limitations of the equipment.

The data recording frequency shall be at least 1 Hz (i.e. one recording every 20 mm of penetration).

Inclination of the cone penetrometer shall be measured. Inclination measurements may be omitted for CPT/CPTUs in drilling mode and in shallow non-drilling mode (<5 m penetration below seafloor).

Guidance and recommendations related to the CPTU pore pressure dissipation tests are provided in 8.4.

The penetration of the cone penetrometer shall be terminated when one of the following conditions are met:

a) the required depth below seafloor has been reached;

b) the maximum thrust or maximum capacity of the measuring system defined by the project specifications has been reached; or

c) inclination of the cone penetrometer approaches or is beyond a limiting value given in the project specifications.

For CPT/CPTUs in drilling mode, reaching maximum thrust capacity at one depth should not prevent subsequent strokes being performed. Reaching maximum thrust capacity is also important information.

Potential damage to the equipment may be a valid reason to terminate a test.

NOTE 4 The achievable penetration length or penetration depth depends on the soil conditions, the allowable penetration force, the allowable forces on the push rods and push rod connectors, the use of a friction reducer and/or push rod casing and the measuring range of the cone penetrometer.

For each test, the differences observed in reference readings and the larger value of output stability obtained before and after the penetration phase of the test shall be used to determine the test category according to Table 4. Tests shall meet all the requirements for each of the measurands for a given test category to be accomplished, where applicable.

NOTE 5 The project specifications can include criteria for avoiding repeating of tests that do not meet the requirements of Table 4. Annex A includes guidance.

Table 4 — Test categories of CPT/CPTU

Test category	Application	Measurand	Difference of reference readings	Output stability
A	Characterisations of seabed with $q_{c;max}$ < 2 MPa	Cone resistance	≤35 kPa	≤11 kPa
		Sleeve friction	≤15 kPa	≤1,5 kPa
		Pore pressure	≤25 kPa	≤8 kPa
B	Characterisations of seabed with 2MPa < $q_{c;max}$ < 60 MPa	Cone resistance	≤100 kPa	≤33 kPa
		Sleeve friction	≤25 kPa	≤5 kPa
		Pore pressure	≤50 kPa	≤16 kPa

$q_{c;max}$ is the maximum value of cone resistance measured during the penetration phase of the test.

The difference of reference readings is according to 8.3.4.

The larger value of output stability according to 8.3.4 applies.

Table 4 *(continued)*

Test category	Application	Measurand	Difference of reference readings	Output stability
C	Characterisations of seabed with $q_{c;max}$ > 60 MPa	Cone resistance	≤200 kPa	≤66 kPa
		Sleeve friction	≤50 kPa	≤8 kPa
		Pore pressure	≤100 kPa	≤33 kPa

$q_{c;max}$ is the maximum value of cone resistance measured during the penetration phase of the test.

The difference of reference readings is according to 8.3.4.

The larger value of output stability according to 8.3.4 applies.

8.3.5 Presentation of test results

The reporting of results from CPT/CPTUs shall conform to 8.2. The net area ratio, *a*, of the cone penetrometer and the end areas of the friction sleeve, determined according to ISO 22476-1, shall be reported for each penetrometer used during the marine soil investigation. Calibration reports, cone penetrometer class, reference readings , output stability, and test category shall be reported. Test category shall be reported for each test, where test category is defined as the combination of IDs of Table 2 and Table 4.

NOTE 1 Examples of possible test categories are 1+B, 1A and 2B.

Measured values of the following parameters shall be reported as functions of depth, with seafloor as zero reference for the parameter values, where applicable:

— cone resistance, q_c;

— pore pressure, $u_2(u_1, u_3)$;

— sleeve friction, f_s;

— inclination of the cone penetrometer, *i*;

— applied thrust to push rod, F_{thrust};

Depth should be derived from penetration length and inclination, *i*, of the cone penetrometer, where values of *i* are available.

Any corrections applied to the measured data shall be reported. Measured values shall be reported in digital (i.e. numerical) format without the application of corrections performed manually, by electronic means or by microprocessors.

Derived values of the following parameters, reported as functions of depth, should include:

a) corrected cone resistance, $q_t = q_c + (1-a)u_2$, where *a* is the net area ratio of the cone determined according to 8.3.3.2;

b) friction ratio, $R_f = f_s/q_c$;

NOTE 2 Sleeve friction is influenced by surrounding water pressure. Since it is not standard practice to measure the pore pressure u_3 above the friction sleeve, the uncorrected sleeve friction f_s is commonly used. A possible correction method for the friction sleeve (to get corrected sleeve friction f_t) is given in Annex A.

c) pore pressure ratio, $B_q = (u_2 - u_0) / (q_t - \sigma_{v0})$,

where:

σ_{v0} is the (estimated) in situ vertical total stress relative to seafloor;

u_0 is the (estimated or measured) in situ equilibrium pore pressure relative to seafloor.

The basis for computation of the σ_{v0} and u_0 profiles shall be reported, where applicable.

NOTE 3 Derived values of additional parameters that can be reported include: net cone resistance, $q_{net} = q_t - \sigma_{v0}$; normalised cone resistance, $Q_t = q_{net}/\sigma'_{v0}$; and normalised friction ratio, $F_r = f_s/q_{net}$.

The following scales should be used for graphical presentation, where applicable:

— depth, 1 scale unit = 1 m;

— cone resistance q_c, (q_t, q_{net}), 1 scale unit = 2 MPa or 0,5 MPa;

— sleeve friction, f_s , 1 scale unit = 0,05 MPa;

— pore pressure, u_2 (u_1, u_3), 1 scale unit = 0,2 MPa or 0,02 MPa;

— friction ratio, R_f, 1 scale unit = 2 %;

— pore pressure ratio, B_q, 1 scale unit = 0,5 units;

— applied thrust to push rod, F_{thrust}, 1 scale unit = 20 kN.

NOTE 4 It can be beneficial to present parameter values at multiple scales when soil parameters values show large vertical variations. For instance, if there are layers of dense sand and clay in the same profile. If the results in clay are to be used for interpretation in terms of soil parameters, it is particularly important to use an enlarged scale in the presentation of test results. It is also important that consistent scale(s) are used.

CPT/CPTU measured values, values for F_{thrust} and, where applicable, derived values shall be reported in digital (i.e. numerical) form.

8.4 Pore pressure dissipation test

8.4.1 General

This subclause covers PPDT performed as part of a cone penetration test CPTU according to 8.3.

The project specifications shall include:

a) type of test according to Table 5;

b) test depth;

c) test duration in terms of termination criteria.

NOTE 1 Increasing test duration can improve the uncertainty of interpretation of test results, particularly for test types PPDT1 and PPDT2.

NOTE 2 Termination criteria can include: a predetermined maximum test duration, (e.g. five hours); a certain percentage of the dissipation of the pore pressure immediately before the penetration interruption, relative to the estimated in situ equilibrium pore pressure; or the change in pore pressure over a given time interval is below a certain value (e.g. 3 kPa per 10 min). Termination criteria can also consider gas in the soil preventing reliable pore pressure measurement, soil heterogeneity or absence of significant penetration pore pressure for interpretation of the test results.

Table 5 — Pore pressure dissipation test types

Test type	Description	Test duration
PPDT1	Pore pressure dissipation for estimation of in situ equilibrium pore pressure	Typically, ≥ 80 % dissipation of the pore pressure immediately before the penetration interruption, relative to the estimated in situ equilibrium pore pressure u_0
PPDT2	Pore pressure dissipation for estimation of coefficient of (radial) consolidation in fine, low permeability soil	Typically, ≥50 % dissipation of the pore pressure immediately before the penetration interruption, relative to the estimated in situ equilibrium pore pressure u_0
PPDT3	Pore pressure dissipation for qualitative indication of soil permeability	Typically, <600 s
PPDT4	Pore pressure dissipation to distinguish between drained, undrained and partially drained soil behaviour during cone penetration	Typically, <60 s

8.4.2 Equipment

Equipment for a pore pressure dissipation test should consist of a piezocone penetrometer and ancillary equipment in accordance with 8.3.

Other probe types may be used, for example a piezoprobe or a ball penetrometer equipped with a pressure sensor, provided the information listed in 8.7.1 is given.

8.4.3 Test procedure

The test procedure shall be as follows:

a) interruption of the push-in penetration of the penetrometer at the required test depth for pore pressure dissipation;

b) recording of pore pressure, cone resistance and sleeve friction versus time until the predetermined termination criterion;

c) resumption of penetration, if applicable.

Data recording should be at 1 Hz or higher during the initial 60 s and can be halved every log (time) cycle thereafter.

8.4.4 Presentation of results

The following information shall be reported:

a) results from the CPTU data acquisition according to 8.3, where applicable;

b) test type according to Table 5;

c) test depth for the filter position of the piezocone penetrometer;

d) filter position of the piezocone penetrometer;

e) method of fixing the position of the piezocone penetrometer during the dissipation phase;

f) pore pressure, u_2, and cone resistance, q_c, versus time and log time, including the values of pore pressure (u_{2i}) and cone resistance (q_{ci}) immediately before the penetration interruption.

Normalized excess pore pressure, U, can additionally be reported versus time and log time, where $U = (u_2 - u_0)/(u_{2i} - u_0)$.

NOTE The position of the filter has a significant influence on the excess pore pressure generated during penetration.

In line with 8.3.2, a filter element location in the cylindrical part just above the cylindrical extension of the cone is recommended. A filter element at location 1 may be used, in which case test results shall be presented for u_1 and u_{1i}, instead of u_2 and u_{2i}.

8.5 Ball and T-bar penetration tests

8.5.1 General

Ball and T-bar penetrometers are intended for deriving intact and remoulded shear strength of very soft to firm clays and clayey silts with a ball (T-bar) resistance $q_m < 500$ kPa.

There are many similarities between ball and T-bar penetrometers. In practice, the ball can be used in both non-drilling and drilling modes, while the T-bar should be use in non-drilling mode only.

The ball and T-bar penetrometers shown in Figure 3 are of a size suitable for mounting directly on standard CPT push rods.

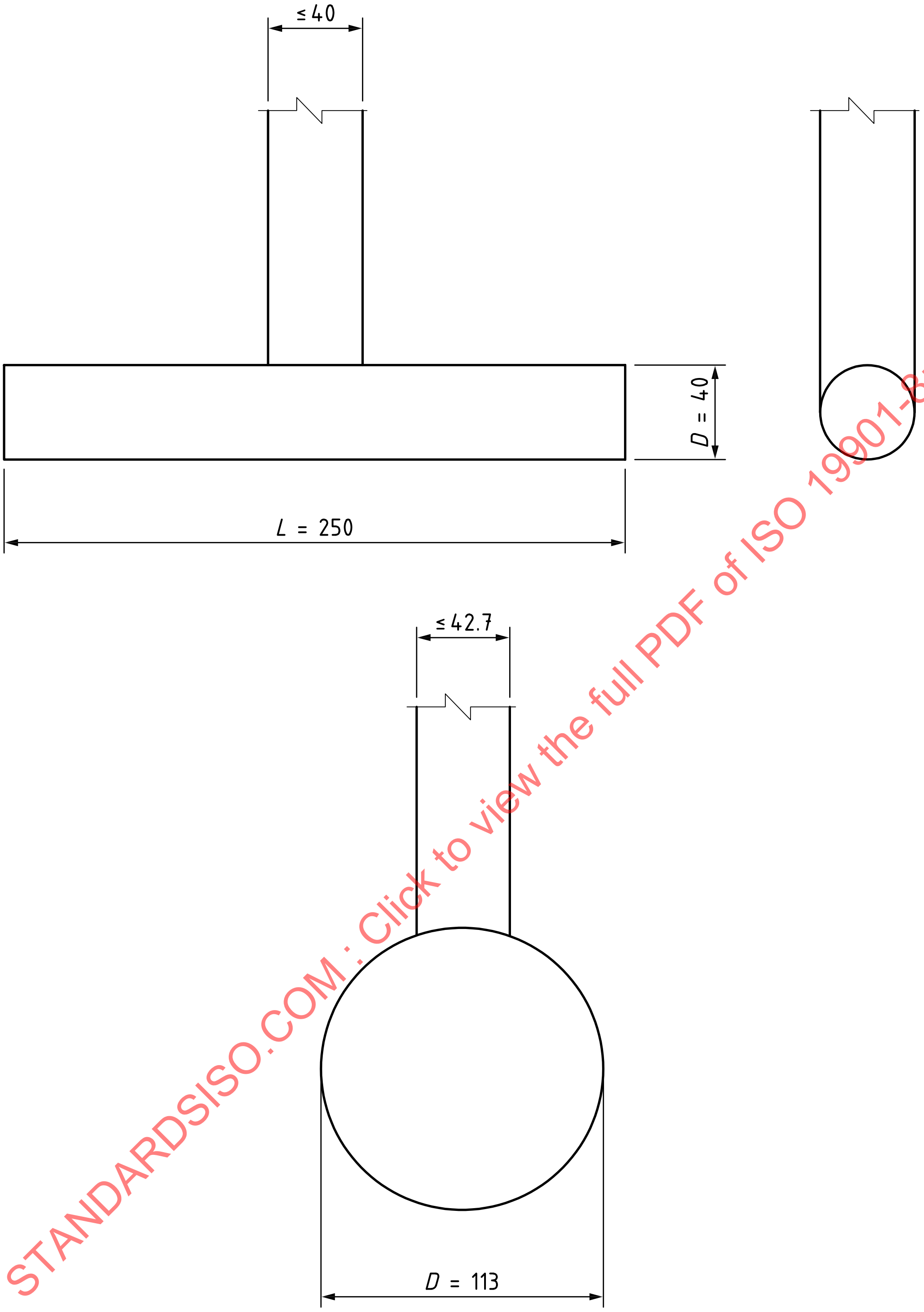
The load sensor for ball (T-bar) resistance shall be positioned within the ball (T-bar) or immediately above the ball (T-bar). The load sensor shall be compensated for possible eccentricity of soil resistance mobilized on the ball (T-bar) penetrometer. For a T-bar penetrometer, the load sensor should also be compensated for a possible torsional component of soil resistance acting on the penetrometer.

NOTE The load cell(s) within a cone penetrometer (without the cone) can be used to measure the soil resistance mobilized on the ball (T-bar) penetrometer.

The ball (T-bar) penetrometer shall be fitted with an inclination sensor with a measuring range of at least ±15° relative to the vertical axis. The inclination sensor may be omitted for tests in shallow non-drilling mode (<5 m penetration below seafloor).

Requirements for the thrust machine, the push rods and the measuring system for penetration should be as given in 8.3.

Dimensions in millimetres



Key

L length of T-bar

D diameter of T-bar or ball

NOTE Upper part of figure shows T-bar. Lower part of figure shows ball.

Figure 3 — Dimensions of 10 000 mm^2 ball and T-bar

8.5.2 Equipment

8.5.2.1 Ball penetrometer

The ball penetrometer (see Figure 3) is a steel sphere attached to a push rod, which is pushed into the soil using CPT/CPTU type deployment systems. The ball shall have a projected area of the penetrometer in a plane normal to the shaft A_p of 2 500 mm^2 (corresponding to a diameter of 56,4 mm) to 10 000 mm^2 (corresponding to a diameter of 113 mm). The diameter of the push rod shall be such that the minimum ratio of the projected area of the ball to the cross-sectional area of the push rod is 7:1, and with no increase in diameter of the connecting push rod for a distance of 10 times the diameter of the rod.

The surface roughness and hardness of the ball penetrometer should be according to the requirements for the friction sleeve of a cone penetrometer provided in 8.3.2.

8.5.2.2 T-bar penetrometer

The T-bar penetrometer is a cylinder, or short bar, attached to a push rod, which is otherwise equivalent to the ball penetrometer (see Figure 3). The T-bar penetrometer used in offshore practice has a nominal diameter projected area of 5 000 mm^2 to 10 000 mm^2 (where A_p = 10 000 mm^2 correspond to a diameter 40 mm and a length of 250 mm , see example in Figure 3). The T-bar penetrometer should satisfy similar requirements as for the ball penetrometer (see 8.5.2.1) in terms of the 7:1 ratio of the projected area of the T-bar to the cross-sectional area of the connecting push rod, and the minimum length of any push rod of reduced diameter. In addition, the T-bar should have a minimum length-to-diameter ratio of 5 and a diameter that is no smaller than that of the connecting push rod.

The surface roughness and hardness of the T-bar penetrometer should be according to the requirements for the friction sleeve of a cone penetrometer provided in 8.3.2.

8.5.3 Calibration and verification of ball and T-bar penetrometers

The ball (T-bar) shall be calibrated for the following measurands:

a) ball (T-bar) resistance;

b) inclination.

Calibration practice can be similar to that described for a cone penetrometer per 8.3.3.2.

Ball and T-bar penetrometer can also be equipped with a pore pressure sensor, in which case it shall also be calibrated.

Test methods and results of verification tests on ambient temperature stability, transient temperature stability and bending influence for the ball (T-bar) penetrometer should be reported. Test methods similar to those for cone penetrometers (8.3.3.2) can be applied. For the T-bar penetrometers, verification of the effects of combined bending and torsion should also be considered. For cyclic testing, consideration should be given to calibration of the ball (T-bar) penetrometer for tensile forces applied to the load sensor for q_m.

8.5.4 Procedures for testing offshore

8.5.4.1 General

The following procedures for testing offshore shall apply:

a) the ball (T-bar) resistance shall be measured during penetration;

b) the rate of penetration derived from the depth sensor shall be (20 ± 5) mm/s. The rate of penetration may be less than (20 ± 5) mm/s immediately before and immediately after a penetration interruption, including specific stationary testing (e.g. PPDT, SCPT), start and end of penetration and add-on of a push rod. These variations on the rate of penetration shall be recorded and reported.

c) Inclination of the ball (T-bar) penetrometer shall be measured. Inclination measurements may be omitted for tests in drilling mode and for tests in shallow non-drilling mode (<5 m penetration below seafloor).

d) readings from ball (T-Bar) resistance sensor, the depth sensor and, where applicable, the inclination and pore pressure sensors shall be recorded at least once per second (i.e. for every 20 mm of penetration).

The project specifications can include requirements for measuring ball (T-bar) resistance during retraction at a nominal rate of (20 ± 5) mm/s.

The project specifications can include requirements for cyclic testing to obtain a remoulded shear strength.

If cyclic ball (T-bar) tests are carried out, these should be done on ball (T-bar) penetration and comprise cycles of minimum up and down distances of at least 0,15 m (0,20 m) or three times the diameter of the ball (T-bar), whichever is greater, at a nominal rate of 20 ± 5 mm/s. Ten full cycles should be carried out, unless no further degradation in ball (T-bar) resistance is evident over three or more cycles.

In some cases, important information can be obtained by carrying out tests at rates of penetration outside those given above. If such tests are carried out, results shall clearly be marked with a note that a non-standard penetration/extraction rate has been used.

Consideration of the proximity of the ball (T-bar) to the seafloor during cycling should be given, for example so as to minimize effects on test results.

Reference readings and values of output stability for ball (T-bar) and pore pressure, where applicable shall be reported before and after the penetration phase of the test. For reference readings, the output of ball (T-bar) resistance and, if applicable, the pore pressure shall be continuously recorded for a period of one minute at a frequency of at least 1 Hz before and after the penetration phase of the test under conditions of no soil pressure and under similar temperature and ambient pressure conditions. For each of these periods, reference readings for ball (T-bar) resistance and pore pressure, where applicable shall be calculated as the mean of the recorded output values. The difference in reference readings for ball (T-bar) resistance and pore pressure, where applicable, shall be between -35 kPa and +35 kPa. Output stability is defined as for CPT/CPTU, in 8.3.4. The variation in output stability for ball (T-bar) resistance and pore pressure, where applicable, shall be ≤ 11 kPa

For seafloor-based drilling and non-drilling systems, reference readings can be made at deck level, at the seafloor or a short-fixed distance above (see A.8 for reference readings for CPT/CPTU). For ball (T-bar) tests during vessel drilling operations, the reference readings shall be at deck level. The output of ball (T-Bar) resistance and, if applicable, the pore pressure of the ball (T-bar) penetrometer at the bottom of the borehole should be compared to theoretically calculated values.

Temperature changes in the ball (T-bar) can cause significant sensor drifts, so it is important to ensure similar temperatures for the ball (T-bar) while taking reference readings before and after the penetration phase of each test.

8.5.5 Presentation of results

The reporting shall conform to 8.2. The results of calibration and verification of the ball (T-bar) penetrometer shall be reported, including the net area ratio of the ball (T-bar), *a*.

Reference readings and values of output stability for ball (T-bar) resistance and pore pressure, where applicable shall be reported, for both before and after the penetration phase of the test.

The test results shall be reported in digital (i.e. numerical) form, consisting of the following:

a) depth below seafloor, expressed in metres;

b) ball (T-bar) resistance, $q_m = F/A_p$ during penetration and extraction (if applicable), expressed in MPa or kPa, where F is the measured axial force on the ball (T-bar) relative to zero at seafloor;

c) pore pressure, u (if measured), during penetration and extraction (if applicable), expressed in MPa or kPa, relative to zero at seafloor;

d) inclination (if measured), expressed in degrees from vertical.

e) applied thrust to push rod, F_{thrust}.

In addition, the net ball (T-bar) resistance should be given according to:

$$q_{ball}\ (q_{T\text{-}bar}) = q_m - [\sigma_{v0} + u_0(1 - a)](A_{sp}/A_p)$$

where

$q_{ball}\ (q_{T\text{-}bar})$	is the net ball (T-bar)resistance
q_m	is the measured ball (T-bar) resistance;
u_0	is the (estimated or measured) in situ equilibrium pore pressure at the mid-height of the ball (T-bar), relative to seafloor;
σ_{v0}	is the (estimated) in situ vertical total vertical stress, relative to seafloor;
A_p	is the projected area of the penetrometer in a plane normal to the shaft;
a	is the net area ratio of the ball (T-bar), $a = A_n/A_{sp,}$ where A_n is the cross-sectional area of the load cell or shaft where pore pressure can act, and A_{sp} is the cross-sectional area of the connecting shaft.

The basis for computation of the σ_{v0} and u_0 profiles shall be reported, where applicable.

For q_m during extraction the effect of the area of the shaft shall be ignored.

The location of the load cell relative to the ball (T-bar) should be reported. This can be useful for the case when the force sensor is well above the ball (T-bar) so the measurement can be adjusted to account for friction on the shaft.

For the reporting of cyclic tests, all cycles should be reported. The net remoulded ball (T-bar) resistance $q_{ball,rem}$ ($q_{T\text{-}bar,rem}$) should be determined as the average absolute values of penetration and extraction resistance (q_{ball}, $q_{T\text{-}bar}$) over the last cycle. If full degradation has not occurred, the report shall clearly mention that the average net remoulded ball (T-bar) resistance from q_{ball} or $q_{T\text{-}bar}$ does not correspond to full degradation. The results of cyclic ball (T-bar) tests should be included in the main graphical plot as well as in an enlarged plot in order to illustrate the results in adequate detail.

In general, test results should be reported with a depth scale of 1 scale unit = 1 m, but for shorter profiles (e.g. for pipeline investigations) an enlarged scale can be used. The scale for presenting ball (T-bar) resistance during penetration and retraction should be selected to suit the soil conditions.

8.6 Seismic cone penetration test

8.6.1 General

A seismic source and a seismic cone penetrometer allow the execution of a downhole or seafloor-based seismic test from which shear wave velocity can be derived, typically vertically propagating horizontally polarised shear waves. The seismic cone penetrometer has one or more seismic receivers that detect shear wave energy generated by a dedicated seismic source deployed at the seafloor. These receivers are integrated with the rest of the CPT/CPTU system.

The requirements of 8.3 shall apply to the CPT/ CPTU part of the SCPT.

8.6.2 Equipment

The geometry including surface roughness and hardness of the CPT/ CPTU part of the seismic cone penetrometer shall be according to 8.3.2. The diameter of the cone penetrometer at the location of the seismic receiver(s) may be greater than that of the sections immediately below the instrumentation package (as recommended by ASTM D7400/D7400). Typical seismic receivers include multi-axial or dual-axial geophones or accelerometers, located either at effectively one point, or at two points in the same tool (dual-element seismic cone penetrometer with receivers separated by a fixed distance, typically 0,5 m or 1 m).

The seismic source can be installed on the seafloor frame used for the cone penetration test or at a different position on the seafloor.

Reversible polarity hammers and vibrator systems can be used as a seismic shear wave source and should be capable of producing repeatable signals. Guidance on assessment of source repeatability is given in A.8.6.

8.6.3 Procedures for testing offshore

Verification of the data recording system before testing should typically include documentation of trigger timing, recording interval, source repeatability and source frequency content.

The project specifications shall include:

a) the type of seismic cone penetrometer (single element or dual element);

b) the test depth or vertical test spacing;

c) termination criteria.

NOTE Shear wave velocity cannot reliably be derived in the upper 2 m to 5 m below seafloor, depending on the system characteristics and site conditions.

Termination criteria can include one or more of the following:

— maximum test depth;

— minimum signal-to-noise ratio combined with a maximum number of signal stacking events;

— termination criteria as per 8.3.4.

Achievable test depth depends on factors such as deployed system characteristics, interaction of the seismic source with the upper seabed, soil conditions at depth and interference from nearby objects.

During testing the horizontal distance between the centre of the footprint of the seismic source and the axis of the push rods at seafloor level shall be determined within a standard uncertainty of 0,2 m.. The horizontal and vertical orientations of the seismic source shall be determined within a standard uncertainty of 20 degrees.

Ground coupling between source and seafloor is paramount for shear wave signal penetration into the ground. Where the source is installed on the seafloor frame, mechanical decoupling of the source and the seafloor frame will minimize transfer of shear wave through the frame and push rods.

The test procedure consists of activating the seismic source and receiving the generated signals at the seismic receivers. This is either done during an interruption in cone penetration or continuously during cone penetration. If cone penetration is interrupted, then multiple activations of a hammer seismic source at a single depth can improve data quality, particularly when deploying a dual polarity seismic source or when applying the ‘stacking method’ according to ASTM D7400/D7400M.

A discrete marker point in the receiver recording shall be generated at the time the seismic source is activated at seafloor, i.e. at the start of propagation of shear waves. The trigger signal and the measurements from the seismic receivers in the seismic cone penetrometer should be recorded as discrete time histories. Data recording should include pre-triggering recording of signals from the seismic receivers. The pre-trigger recording interval can be selected on a test-by-test basis.

The recommended maximum sample interval is 0,025 ms (which corresponds to a minimum sampling frequency of 40 Hz). Guidance on repeatability and uncertainty of the triggering mechanism and sampling frequency of the recording system is provided in Annex A.

Data recording shall cover unfiltered data from each of the seismic receivers, downhole test depth, application of data or signal filters applied during acquisition, applied signal gain, and sampling frequency of the seismic receivers. Recorded signals shall be reported in digital (i.e. numerical) format without the application of filtering and corrections performed manually, by electronic means or by microprocessors.

8.6.4 Presentation of results

Reporting of results shall be according to 8.3.5 and shall additionally cover:

a) type and description of the seismic source, the seismic receiver(s) and the trigger system;

b) difference in depths between the seismic source and the position(s) of the seismic receiver(s);

c) the horizontal distance and orientations (horizontal and vertical) of the seismic source relative to the axis of the seismic cone penetrometer as defined in 8.6.2;

d) primary shear wave component for each seismic receiver at each test depth or as a continuous record;

e) derived value of shear wave velocity, v_s, and the depth interval over which it has been derived, including:

— assumed travel path for the seismic waves;

— the applied method of signal picking and matching.

f) a comparison of the derived shear wave velocity with at least one CPT based shear wave velocity correlation.

Guidance on quality assessment of the reported data from the seismic receiver system is provided in A.8.6.

NOTE 1 A travel path is not always a straight line for materials with an abrupt change in density or stiffness, and in such cases Snell's law of refraction can be used. With small offset distances relative to depth below seafloor (e.g. 2 m offset and 10 m depth below seafloor this becomes less important.

NOTE 2 Data recorded by a seismic receiver can include influence from for example energy absorption in the ground, energy scattering, reflection/refraction of the seismic wave and seismic waves travelling along layer boundaries.

8.7 Other in situ tests

8.7.1 General

In addition to the in situ tests described in 8.3 to 8.6, other appropriate tests can be needed or proposed depending on the objectives of the marine soil investigation. Such tests shall be described in the project specifications. Guidance is provided in A.8.7.

Reporting of results shall be according to 8.2 and shall additionally cover:

a) description of equipment and purpose of test;

b) geometry of equipment;

c) results of calibration and verification of sensors and geometry, as applicable, that meet the requirements of ISO/IEC 17025;

d) data acquisition system, with a statement on resolution of measured results;

e) summary of test procedure.

9 Sampling

9.1 Purpose and objectives of sampling

The main purpose of sampling is to obtain soil material of all significant layers suitable for description and laboratory testing. The objectives of sampling are to provide samples according to project specifications and scope of work.

The selection of the samplers and deployment modes for particular soil conditions shall be considered in relation to the objectives of the sampling, as it will affect the sample quality. See 10.6 and Table 9 for specific guidance on intact samples and see A.9 for specific guidance on selection of sampling equipment (applicability classes).

The project specifications can include additional requirements on minimization of sample disturbance when sample quality is important, for example for obtaining good to high quality intact or undisturbed samples.

A lower sample quality can be appropriate if the primary objective of the sampling relates to cases such as:

— soil type confirmation for interpretation of in situ tests;

— continuous coverage of the soil vertical profile (soil sample disturbance is a lesser issue);

— to obtain a large sample volume (volume is more important than quality).

In such cases, other types or variations from standard equipment can be preferable (e.g. much larger diameter or greater length of sample tubes).

9.2 Sampling systems

Drilling mode deployment systems utilize an iterative process to progressively recover discrete soil samples from the seabed, with a borehole progressed via rotary drilling between sampling runs. Non-drilling mode deployment systems progress samplers in a single stroke into the seabed, with sampling being terminated either at the point of maximum stroke of the tool, at a predetermined depth, or due to refusal associated with local seabed obstructions and/or accumulated penetration resistance.

The type of equipment selected generally governs the maximum depth from which a sample can be recovered, as well as the quality and recovery ratio of sample that can be achieved. The project specifications should address that the selected method of deployment will have the capacity not only to penetrate to target depth, but also to acquire samples of sufficient quality to meet the objectives of the marine soil investigation.

9.3 Selection of samplers

9.3.1 General

The choice of samplers shall be made with consideration to the expected soil conditions, as well as the type of laboratory testing to which the samples will be subjected. The project specifications can provide preliminary guidance on appropriate sample tubes. The guidance can include instructions on on-site decisions to be made as further information becomes available during the marine soil investigation.

In many cases, marine soil investigations are undertaken at previously unexplored locations, where preliminary knowledge of the geology is based on the desk study and the results of the geophysical investigation. In such cases, consideration should be given to mobilizing a range of samplers, capable of sampling a range of soil types.

Use of core catchers when appropriate shall be considered to avoid loss of sample, particularly in coarse soil or very coarse soil.

NOTE In some cases, a core catcher can cause no or partial sample recovery, for example when using a core catcher in low strength soil close to seafloor.

The length of sample that can be obtained using a sampler is dependent on:

— the geometry, dimensions and characteristics of the sampler;

— the soil type;

— the available penetration force and how this is applied using the deployment equipment.

The quality of the sample is influenced by sample tube geometry.

The following information should be reported for the samplers, where applicable:

a) cutting shoe angle;

b) inside and outside diameters;

c) inside clearance;

d) maximum sample length;

e) whether a piston is used or not;

f) inside liners or stocking;

g) core catching system;

h) steel material specification;

i) method of sealing sample tubes if samples are temporarily stored at seafloor and if samples are not to be extruded offshore.

Stainless steel or other non-corrodible material shall be used to seal the samples within the tubes if they are not extruded offshore. The method of sealing and selection of sealing material shall aim at avoiding changes in water content.

Sample tubes should not be re-used unless they have been cleaned and checked for damage. Tubes with damaged tips shall be repaired to the original standard or discarded.

9.3.2 Drilling mode samplers

For samplers deployed in drilling mode, a variety of samplers is available. A general guide to sampler types is presented in Table A.6.

To cover the expected range of soil types, the project specification shall specify the available systems (e.g. for taking piston, push and hammer samples). For very soft to stiff clays, the order providing the best sample quality usually is as follows:

a) piston sampler;

b) push sampler, thin-walled;

c) push sampler, thick-walled;

d) rotary core sampler;

e) percussion/vibratory sampler;

f) hammer sampler.

Where applicable, the sample tube penetration, stroke, number of hammer blows, and hammer weight shall be reported for each sample.

In some cases, especially in very stiff clays and cemented soils/weak rocks, the use of a rotary core sampler can give the best quality.

In order to avoid highly disturbed soil below the drill bit, a piston sampler can be pushed through this material and sampling can start at a lower level.

Retrieval and handling of drilling-mode samplers should be undertaken in such a way that any shock or vibration is minimized.

Specifications for drilling mode sampling equipment vary considerably across the industry, and specific details of the proposed sampling equipment shall be considered for final selection. Unless given otherwise in the project specifications, the information presented in Table 6 shall be incorporated in reporting of the results of the marine soil investigation.

Table 6 — Required information for drilling-mode samplers (downhole samplers)

Sampler type	Diameter and length of sample	System description including driving mechanism	Maximum driving force available	Method for measuring penetration
Piston sampler	√	√	√	√
Push sampler, thin-walled	√	√	√	√
Push sampler, thick-walled	√	√	√	√
Rotary core sampler	√	Incl. bit type and dimensions	Torque and static push	√
Percussion/vibratory sampler	√	√	Frequency and energy	√
Hammer sampler	√	√	Hammer mass, fall height and winch characteristics	√

A hammer sampler should not be used as an SPT tool for estimating relative density of sands from number of blows.

NOTE √ denotes information to be documented.

9.3.3 Non-drilling mode samplers

Sampling equipment deployed via non-drilling methods can be appropriate where there is a reasonable degree of confidence that target depths can be reliably achieved in the geology expected. Sampling equipment deployed using such systems can range from sophisticated seafloor-founded push frames, capable of recovering high-quality samples, to very simple seafloor samplers deployed with limited capacity to penetrate as well as limited ability to acquire high quality samples. With a few exceptions, all these systems are generally limited to penetrations less than 25 m below seafloor. However, penetrations in excess of 40 m are possible using specialist equipment in very soft seabed.

Samplers deployed in non-drilling mode provide limited options, and most devices are ideally suited to soft, fine soils that offer little resistance to sampler penetration. Deployment of such tools in very stiff/dense soil conditions result in limited depth penetration and sample recovery, and with increased possibility of damage to the equipment. A general guide to sampler types available for non-drilling mode is presented in A.9.3.3 and Table A.7.

The following non-drilling mode samplers are frequently used in the industry:

— piston core sampler with fixed reference to seafloor;

— piston core sampler without a fixed reference;

— gravity core sampler (without piston);

— vibrocore sampler;

— box core sampler;

— grab sampler.

The box corer shall be fitted with a device to prevent early triggering during overboarding of the corer.

The trip release mechanism of the piston corer shall be fitted with a safety device to prevent early triggering of corer during overboarding of equipment.

Further information on the use of these samplers is provided in A.9.3.3.

Specifications for non-drilling mode sampling equipment vary considerably across the industry. Specific details of the proposed sampling equipment shall be considered for final selection of equipment. A summary of specifications is presented in A.9.3.3.8 and Table 7. Unless given otherwise in the project specifications, the information presented in Table 7 shall be incorporated in reporting of the results of the marine soil investigation.

Table 7 — Required information for non-drilling mode samplers (seafloor sampling equipment)

Sampler type	Sampling depth below seafloor (max.)	Geometry and dimensions of cutting shoe	Inside and outside diameters of core barrel and liner	Weights and lengths available	Method of measuring penetration	Specification of driving mechanism	Vertical reference of piston during sampling	Core catcher or sample retainer arrangement	Additional information can be required on
Piston core sampler with fixed reference to seafloor	√	√	√	√	√	√	√	√	Non-return valve
Piston core sampler without fixed reference to seafloor	√	√	√	√	√	√	√	√	Non-return valve
Gravity core sampler	√	√	√	√	√	√	n/a	√	Non-return valve
Vibrocore sampler	√	√	√	√	√	√	√	√	Frequency and energy
Box core sampler	√	n/a	n/a	√	√	√	n/a	√	Sub-sampling and wash-out prevention
Grab sampler	n/a	n/a	n/a	√	√	√	n/a	√	Release mechanism, max. volume of sample and wash-out prevention
NOTE √ denotes information to be documented.									

9.4 Sample recovery considerations

Sample loss is observed when the length of sample acquired (as measured on deck level) is less than the penetration of the sampler. This can be positive (material is lost) or negative (material has expanded or recovered from a previous sampling operation in the case of rotary core drilling). Reasons for differences between sample recovery and sampler penetration are typically sample loss, compression, stretching or expansion. Sample loss also affects data depth uncertainty as described in Clause 6.

Sample loss can be particularly prevalent in non-drilling mode, where long samples are generally recovered in a single sampling stroke.

Sample loss can occur due to one or more of the following events:

— loss of sample from the bottom end of the sampler during retrieval;

— densification of sample within the tube/liner due to shock or vibration;

— loss and/or segregation of very soft soil due to fluidization;

— settling of soil when the inside clearance ratio C_i (see A.9.3.2.2) of the sampler is greater than zero;

— soil expansion due to the presence of reactive clays, dissolved gases and/or stress release;

— plugging of the cutting shoe during sampler penetration;

— piston accelerations that are out of phase with sampler penetrations;

— washing out/erosion of sample due to excessive exposure to drilling fluid during rotary core drilling;

— loss of sample due to soil type (e.g. gravel, loose coarse soils, frozen soils, and rock).

Identification of the exact cause of sample loss is often difficult, and in many cases only an estimate can be made of the depth interval where sample loss took place. Gaps in sample continuity can be attributed to geology, a fault of the sampler, poor control of the sampler itself during penetration or poor drilling resulting in disturbance of soil in the sampling zone. For this reason, maintenance of detailed sampling records, where practicable, can provide a useful additional tool for the identification of problematic areas in which sample loss can have occurred.

In addition to sampling records, the superposition of nearby in situ test data can also assist in making an appropriate estimate of where sample loss has occurred. High quality UUHR geophysical investigation data (e.g. from sub-bottom profilers) can also assist in evaluation of where sample loss has occurred. If the location of sample loss/extension is unknown, the sampling record according to 9.5.1 shall indicate this and shall show sample loss/extension at the bottom of the sample.

For samplers with no fixed seafloor reference, the measurement of penetration can be uncertain and sample loss or recovery ratio can be difficult to estimate. Depth uncertainty class Z5 often applies.

9.5 Handling, transport and storage of samples

9.5.1 General

Sample handling, transport and storage practice shall suit the purpose of sampling .

Operating and handling procedures that provide a demonstrated consideration of the sample history, from initial setup of the sampler prior to sampling to delivery to the onshore soil-testing laboratory, shall be reported. Sample tracking is important. Each sample shall have a unique ID, either by manually logging (recording) the borehole number, sample number, depth, etc., as specified, or by an ID link with a database (e.g. bar coding, RFID). Depth reference for the sample shall be top of sample. For non-drilling mode samplers all observations providing information on penetration depth shall be reported.

For each sampling operation made by drilling mode and non-drilling mode samplers a sampling record shall be prepared and maintained. The sampling record shall include the following information:

a) ID of borehole and sample;

b) date of sampling;

c) location coordinates, including coordinate reference system;

d) water depth;

e) depth below seafloor to the top of the sample;

f) type of sampler;

g) dimensions of sampler;

h) length or volume of sample collected;

i) whether sample is extruded or sealed in tube or liner, etc.

The sampling record shall be completed as soon as practicable following sample acquisition.

9.5.2 Offshore sample handling

Following acquisition, the sample shall be handled in such a way that sample disturbance is minimized.

For samples suspected to consist of contaminated material, special precautions shall be taken, and expert guidance shall be sought depending on the state and type of any such material.

For drilling mode samples, any 'cuttings' and drill mud shall be removed from the top of the sampler before the total length of sampled soil is measured. Samples recovered in long plastic liners can be carefully cut into smaller lengths of 0,5 m to 1 m, where the material strength permits.

The decision whether or not to extrude soil samples on site is a function of the material, the purpose of sampling and the requirements for offshore testing. Generally, only samples of fine soil and samples that will remain in a relatively intact state after extrusion should be extruded on site, although sand samples can also be considered for extrusion. Other samples should be retained within the sample tubes. Dense fine-sand samples and sand samples containing silt or clay are normally best kept in the sample tubes.

NOTE For samples retained in sample tubes there can be set-up effects over time, which can further reduce sample quality during sample extrusion.

The top and bottom of sample or sample segments shall be clearly marked.

If an extruder is used, the extruder shall provide a continuous rate of displacement (nominal minimum speed 10 mm/s) and shall have an adaptor custom-fitted to the dimensions of samples recovered. The extruder shall not impart excessive vibrations to the sample. The extruded sample shall be "received" in a stable cradle like support with as little friction as possible (e.g. by an aluminium foil interface that follows the sample).

If samples are suspected to contain gas, special precautions, particularly in the field of personnel safety, shall be considered.

A suggested procedure for handling of extruded samples is given in A.9.5.2.1.

For tube samples not extruded offshore, soil description and index tests according to Annex B can be performed at the sample ends. A suggested procedure for handling of these samples is given in A.9.5.2.2.

Samples recovered by rotary core sampling shall be carefully handled, logged, tested (where applicable) and prepared for transport. Guidance on this issue is provided in A.9.5.2.3.

Samples acquired by grab or box core methods shall be considered for sub-sampling or otherwise shall be logged and handled according to the project specifications. Guidelines and suggestions are provided in A.9.3.3.7 for grab samples and in A.9.3.3.6 for box core samples.

Samples retrieved using a vibrocore sampler should be stored in an upright position (following cutting into sections of no less than 1 m), sealed and labelled in accordance with A.9.5.2.2 with the top of the tube clearly identified to maintain the ability to interpret stratigraphy in the onshore lab after transportation.

9.5.3 Offshore storage

Offshore storage of intact samples shall avoid exposure to high temperatures, freezing, chemical and moisture changes, and minimise vibration or impact/shock. All intact samples, which have not been extruded in the field, shall be stored vertically in the same orientation as in situ.

All intact tube samples should be stored in a temperature-controlled environment targeting in situ temperature such that extremes of temperature are avoided. The temperature range should be between 5 °C and 25 °C.

Further guidance on offshore storage is provided in A.9.5.3.

9.5.4 Onshore transport, handling and storage

Appropriate procedures for sample transport, handling and storage prior to arrival at the onshore laboratory shall be implemented. In general, conditions of temperature, moisture, vibration and shock shall be controlled, and packing shall be sufficiently robust to avoid damage to samples. Freezing shall be avoided. Measures shall be put in place to ensure the samples do not experience freezing conditions during air freight or in cold climate transport. The temperature should target in situ temperature but not range beyond 5 °C and 25 °C (lab conditions).

ISO 22475-1 provides guidance on handling, transport, onshore storage of samples and sampling records.

Personnel involved in transportation of samples shall be aware of the delicate nature of the samples and shall exercise ‘handle with care’ to minimize disturbance to the samples during uploading, transporting and offloading.

Generally, long-term storage of about 6 months to 12 months should be considered for a typical offshore project. A room with constant temperature and humidity should be used for long-term storage of the samples in order to minimize alteration of soil properties.

10 Laboratory testing

10.1 General

This clause and Annex B cover testing of soils performed in laboratories both offshore and onshore.

The laboratory programme shall be appropriate to the general objectives of the marine soil investigation and the project specifications. The laboratory programme should be developed by an experienced geotechnical engineer or related specialist.

Laboratory tests shall be performed according to international or national standards. Tests may be performed in accordance with recognized industry standards or codes or other industry recognized procedures. Method statements shall be documented.

Annex B provides procedures for conducting common laboratory tests, with a primary focus on laboratory testing of saturated soils. It does not cover testing of contaminated soils.

Recommendations presented in this document are primarily for testing of conventional soils, such as siliceous sands and clays. Samples selected for a particular laboratory test should match the intended scope of that test. Consideration should be given to alternative and supplementary recommendations when performing marine soil investigations in other types of soils, such as the examples presented in Table A.3.

This document does not cover details of laboratory testing of rock. References to standards for classification and laboratory testing of rock are provided in B.12.

Many aspects of laboratory testing of soils, such as instrumentation, data acquisition, calibrations, corrections, soil preparation and evaluation of sample quality, are common to a variety of tests. Background information for such common topics is presented in 10.3 to 10.6. Some specific test recommendations are given in the relevant subclauses in Annex B.

The applicability of data from tests that require the use of intact samples is significantly influenced by sample quality. Sample quality should be evaluated and reported whenever possible.

10.2 Project specifications

The relevant standards and procedures to be used for the laboratory test programme and the form of presentation and transmission of results, such as tables and figures, should be given in the project specifications (see 5.3). If not given in the project specifications, then the standards and procedures described in Annex B should apply.

Many of the tests presented in Annex B refer to items that should be given in the project specifications.

The project specifications should define the responsibility for determining each of the relevant project specification items for a given test. If the project specifications exclude definition of such responsibility, then responsibility is typically agreed between client and contractor as part of a project execution plan. A common arrangement for laboratory items is a contractor proposal that is reviewed/updated by implementation of client comments.

10.3 Presentation of laboratory test results

Reporting of the laboratory test results shall include the reference and title of the standard (or practice) applied for each of the laboratory tests conducted. Where a standard (or practice) covers more than one method, then the selected method shall be reported.

For each test specimen, presentation of laboratory test results (including plots and tables) shall include:

a) the site location;

b) the specimen identification (e.g. borehole or box core number, sample number, section number);

c) the specimen depth.

Other relevant test and specimen information, where applicable, should include:

a) specimen dimensions;

b) soil density or unit weight (total and dry) from the following phases: initial, after consolidation prior to shear, final;

c) void ratios from the following phases: initial, after consolidation prior to shear, final;

d) water contents from the following phases: initial, after consolidation prior to shear, final;

e) degree of saturation immediately before the start of the primary test phase;

f) specific gravity of soil grains (density or unit weight of solid particles);

g) maximum and minimum index dry densities;

h) plasticity characteristics (Atterberg limits);

i) estimate of the in situ effective vertical stress;

j) reconstitution procedure in case of testing reconstituted specimens;

k) a photograph of the test specimen split open.

Any deviations from the test requirements (e.g. insufficient sample available to perform particular tests, lack of intact samples, test run on specimens that were dimensionally out of standard) shall be identified and reported.

Any relevant comments to the general quality of the tested samples and test results should be reported.

10.4 Instrumentation, calibration and data acquisition

Laboratory testing should be conducted in accordance with ISO 10012 and ISO/IEC 17025. The laboratory should conform with ISO/IEC 17025 for the tests included in the project specifications, where practicable.

NOTE 1 Accreditation can be impracticable for some of the scope included in the project specifications. Example situations include site laboratories and test methods that are uncommon for marine soil investigations.

The instrumentation for laboratory testing can include mechanical devices (e.g. dial gauge, pressure gauge, proving ring) and electronic devices (e.g. displacement transducer, pressure transducer, load cell). Recommendations on the frequency of calibration of instrumentation, commonly every 6 months to 12 months, are given by standards such as ISO 10012 and ASTM D3740. Calibration data shall be available for the tests included in the project specifications, where applicable. The procedures and equipment used to acquire the calibration data and the calibration frequency shall be documented.

NOTE 2 Procedures for calibration are given in Head and Epps (2011)[125] and ASTM D6027 for linear displacement transducers.

Instrumentation selected for specific tests shall meet the resolution, uncertainty and capacity required for the measurement being made, where such resolution, uncertainty and capacity are given in the relevant standard or project specifications. Resolution, uncertainty and capacity largely depend on the type of test being conducted, target displacement, pressure or force levels, and soil strength and stiffness. Uncertainty and reading frequency recommendations for laboratory tests that make extensive use of instrumentation (e.g. triaxial, consolidation and direct shear tests) are given in the specific test subclauses in Annex B.

Annex B include recommendations for corrections that should be applied to test data (where relevant) to account for apparatus deformation, changes in specimen area, piston friction, filter paper resistance, membrane resistance and temperature. Such corrections are most commonly applicable to consolidation, triaxial, DSS, DS and resonant column testing. The procedures and equipment used to determine these corrections should be reported.

10.5 Preparation of soil specimens for testing

10.5.1 Minimum sample size and specimen dimensions

Standards cited in this subclause and Annex B can include minimum values for sample mass and specimen dimensions for most of the commonly conducted laboratory tests.

10.5.2 Preparation of disturbed samples and soil batching

Soil samples shall be used in their sampled state and shall not be dried prior to testing, unless otherwise specified in the project specifications. If drying is required, options include air and oven drying. The method of drying used shall be reported.

Aggregations of particles should be broken down in such a manner as to avoid crushing of individual particles. The method used should be no more severe than that applied by a rubber-tipped pestle.

Disaggregated soil should be thoroughly mixed before subdividing. Subdivision of disaggregated soil should be conducted using methods such as riffling or cone-and-quartering. This procedure should be repeated until a representative sample of the specified minimum mass is obtained for use as a test specimen.

If it is necessary to remove oversized particles to prepare a test specimen, the size range and the equivalent dry mass of the oversized material shall be reported.

In case batching of coarse or very coarse soil is required, the procedures for batching shall be reported. Storage and subsampling of a batch sample should ensure that specimens for testing are representative for the batch. Reporting shall include description of procedures for selection of suitable samples, drying of material, mixing and validation of batch homogeneity.

10.5.3 Preparation of intact specimens (fine soils)

The representativeness of results from tests that require intact specimens (e.g. consolidation, triaxial, direct simple shear, direct shear, permeability tests) is significantly influenced by sample quality. Selection of intact samples for testing should be guided by all information that is available on the potential quality of the samples. X-rays and CT scans (see B.2.4) provide valuable non-destructive visual information on sample quality and can be used to guide selection of test samples. Quantification of sample quality for low to medium over-consolidation ratio clays can be conducted using the a posteriori ε_{vol} or $\Delta e/e_0$ methods described in 10.6. Photographs of the sample/specimen can be helpful as well.

Any unused portions of an intact sample that can be needed for future testing should be properly re-sealed, labelled and returned to storage. Guidelines for sample sealing and storage are given in A.9.4 and in ISO 22475-1.

Trimming of intact specimens shall be conducted in such a manner as to minimize disturbance and loss of water. Specimen handling shall be kept to a minimum. Where possible, the specimen should be supported on a rigid surface. Handling and transfers of the specimen should be conducted using rigid support devices (e.g. plates, cylinders).

Trimming of specimens for triaxial, consolidation, DS and permeability tests should be conducted using suitably dimensioned trimming devices, together with sharp and clean trimming tools (e.g. wire saw, scalpel, knife, straightedge). Suitable trimming devices for cylindrical specimens include a trimming lathe and turntable or cutting ring. For consolidation specimens, the cutting ring can be part of the confinement ring. Cutting rings should have sharp edges and a highly polished inner surface coated with a low-friction lubricant (e.g. low viscosity silicone oil). Further details concerning sample trimming equipment are presented in the relevant test subclauses in Annex B.

Special procedures typically apply to trimming and set-up of specimens having undrained shear strength s_u less than about 5 kPa. Special procedures and equipment used should be reported.

10.5.4 Laboratory-prepared compacted and reconstituted specimens

10.5.4.1 Compacted specimens

Soil samples can be compacted to form a test specimen, either by using a specified compacting force at a specified water content or by achieving a specified dry density at a specified water content. Compaction options include tamping, kneading, ramming, rodding, vibration and static loading. Specimens for consolidation, triaxial, DSS, DS and permeability tests can be compacted into a suitable mould that is either larger than or the same size as the required test specimen. If the mould is larger than the test specimen, the compacted sample should be removed from the mould and trimmed to the required test specimen size using the procedures described in 10.5.3 for intact samples. The formation of voids within the compacted sample or specimen should be avoided. Samples containing clay should be left to stand for at least 24 h at the required water content for the test. The compaction equipment and sample preparation procedures shall be reported.

10.5.4.2 Specimens of coarse soil

Reconstituted samples and specimens of sands can be prepared using a variety of methodologies. The material can be wet, dry or moist, and can be poured in the mould or placed using air or water pluviation as well as by gently using a spoon or a funnel. It can be placed or poured into the mould in stages. The specified densification force can be applied using tamping, tapping, rodding or vibration. Commonly applied reconstitution methods are air and water pluviation methods along with moist tamping methods applying under-compaction of layers, potentially modified to Ladd's under-compaction method[134]. Table 8 summarises some advantages and disadvantages related to each of these methods.

NOTE 1 None of the methods listed in Table 8 can consistently replicate marine deposition and post-depositional processes.

Table 8 — Advantages and disadvantages of different methods for preparing reconstituted coarse soil samples and specimens

	Moist tamping	Air pluviation	Water pluviation
Advantage	Wide range of void ratios can be obtained. Good density control. Less sensitive to particle segregation.	Wide range of void ratios can be obtained. Method is understood as the least prone to soil degradation.	Partially simulates natural alluvial / fluvial / marine deposition. Lower void ratios can be obtained compared to air pluviation.
Disadvantage	Particle crushing can occur at low void ratios.	Sensitive to pouring rate/drop height. Particle segregation can occur.	Layering and particle segregation can occur.

The appropriate reconstitution method shall be selected having the in situ conditions, such as depositional environment, grain sizes and density, in mind in order to replicate in situ conditions as good as reasonably practicable. No reconstitution procedure can be expected to be universally applicable for all sands.

Segregation and crushing of grains (especially for carbonate sands) should be avoided to obtain as homogeneous a specimen as possible. Specimen homogeneity can be evaluated using X-ray scan or by splitting the specimen into subsections and comparing the subsection densities. Specimens for consolidation, triaxial, direct simple shear and permeability testing shall be reconstituted directly into the test specimen mould, including a membrane where applicable. The equipment and procedures used for reconstituting coarse soil samples or specimens shall be documented.

Pre-conditioning or pre-shearing of reconstituted sand specimens can reduce seating and arching effects (particularly for DSS specimens). In the pre-shearing stage, a limited number of low-amplitude cycles (e.g. 400 cycles) can be applied to the specimen. Pre-conditioning can also simulate small-amplitude loading cycles resulting from dynamic/cyclic environmental and mechanical loading.

The dry density of the specimen after the consolidation phase in a triaxial test or in a DS test shall be obtained with a standard uncertainty of ≤1 % of the specified dry density for an estimated corresponding relative density of equal to or greater than 60 %. For an estimated relative density of less than 60 %, the dry density of the specimen after the consolidation phase in a triaxial or DS test should be obtained with a standard uncertainty of ≤1 % of the specified dry density.

NOTE 2 Achieving a standard uncertainty of <1 % of the specified dry density after the consolidation phase can be more challenging for an estimated relative density of less than 60 %. The ability to achieve the correct dry density depends on the grain size distribution, mineralogy, and fines content of the soil, reconstitution method, and the maximum consolidation stress. Achieving the correct dry density can require trial-and-error testing.

10.5.4.3 Specimens of fine soil

Reconstituted samples or specimens of fine soils should be prepared by thoroughly mixing the soil to form a homogeneous slurry. The soil should be prepared starting from the natural water content and adding water such that the slurry water content is greater than approximately 1,3 times the liquid limit.

The mixing water used to prepare the slurry can be that of the appropriate ionic content, de-ionized, or distilled. The water content should be high enough to allow the slurry to be poured into a consolidation cell. Consolidation of the reconstituted sample or specimen should be conducted in a manner similar to that of the consolidation test as described in B.3. Initial loading should be small enough to allow the specimen to stiffen in order to prevent extrusion of material upon additional loading. The final consolidation stress should be sufficient to allow the sample to be extruded from the consolidation cell and handled for sealing and subsequent trimming of test specimens. The equipment and procedures used for preparing reconstituted fine soil samples or specimens shall be documented.

NOTE Laboratory tests on samples of reconstituted fine soil can be used to derive intrinsic soil properties that are independent of soil stress history and structure. The results can be used as frames of reference for the behaviour of natural soils (see Burland (1990)[108], Colliat et al. (2011)[116] and Cottechia and Chandler (2000)[117]).

10.5.5 Preparation of remoulded samples

The procedures used for preparing remoulded samples shall be reported.

Remoulding of fine soil can be achieved by thoroughly mixing, squeezing and kneading the sample while minimizing loss of water. This can be achieved by placing the sample in a sealed plastic bag prior to remoulding.

The energy required to remould a fine soil depends largely on its water content relative to its liquid limit. Soils with low water content and low liquidity index are stiffer and more difficult to remould than those with water content greater than the liquid limit. Thus, greater uncertainty can exist in how thoroughly remoulded very stiff samples are in comparison to soft, high liquidity index samples. A check of the thoroughness of remoulding can be performed by periodically conducting a strength index test (e.g. fall cone test) on the sample during remoulding until a constant value of strength is obtained.

A remoulded test specimen can be prepared by working the remoulded soil into an appropriate mould. This should be conducted as quickly as possible, to minimize a change in water content, and to minimize entrapping air.

10.6 Evaluation of intact sample quality

While no definitive method exists for determining the quality of intact samples, valuable information can be obtained using the following qualitative and quantitative methods.

Qualitative (visual) assessment of sample quality can be made by examination of sample X-rays or CT scans as described in B.2.4. Petrographic examination of soil fabric is another method, which is particularly useful in assessing the amount of disturbance in fine, fragile calcareous soils.

Quantitative assessment of sample quality for intact, low to medium OCR clays can be made by measuring volume change at the estimated in situ stress state (σ'_{v0}, σ'_{h0}) during laboratory consolidation. The normalized sample quality parameter $\Delta e/e_0$ of Lunne et al. (2006)[140] is computed as:

$$\Delta e/e_0 = \varepsilon_{vol}(1 + e_0)/e_0$$

where

Δe is the change in void ratio;

e_0 is the void ratio of the prepared specimen;

ε_{vol} is the volumetric strain (= $\Delta V/V_0$) from reconsolidation to (σ'_{v0}, σ'_{h0});

σ'_{v0} is the best estimate in situ vertical effective stress;

σ'_{h0} is the best estimate in situ horizontal effective stress.

For intact clay specimens, laboratory test methods that include a consolidation phase (e.g. IL oedometer, CRS oedometer and anisotropic consolidation phase of strength tests, such as triaxial and DSS) shall include a specimen reconsolidation phase that allows the computation of $\Delta e/e_0$ and ε_{vol}. The requirements for a specimen reconsolidation phase shall not apply if such a phase will have adverse effects on the primary data intended to be acquired by the laboratory test method. Both the practice details for the reconsolidation phase, estimates of in situ effective stresses, and computed values of $\Delta e/e_0$ and ε_{vol} shall be reported. The intact sample quality parameter can be determined using Table 9 for the method of Lunne et al. (2006)[140]. An alternative method is given by Terzaghi et al. (1996)[161].

Table 9 — Evaluation of sample quality for low to medium OCR clays using Lunne et al. (2006)

OCR	$\Delta e/e_0$ at σ'_{v0}			
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
Sample quality	1 (very good to excellent)	2 (fair to good)	3 (poor)	4 (very poor)

The anisotropic consolidation stresses $\sigma'_{vc} = \sigma'_{v0}$ and $\sigma'_{hc} = \sigma'_{h0}$ can be directly applied in an anisotropically consolidated triaxial (reconsolidation) test, whereas in tests with rigid confinement (e.g. incremental load oedometer, constant rate of strain, and the common form of direct simple shear) only σ'_{vc} is known, and an unknown value of σ'_{hc} develops in the specimen. In such tests, the assumption of $\varepsilon_{vol} = \varepsilon_a$ (axial strain from reconsolidation to σ'_{v0}, σ'_{h0}) may be applied.

NOTE 1 The intact sample quality criteria in Table 9 are not valid for data from IL tests that use long load durations (e.g. 24 h) because of added secondary compression. The sample quality criteria in Table 9 or the ones given by Terzaghi et al. (1996)[161] can be used if the IL test was conducted using relatively short load durations (e.g. 1 h to 3 h as suggested by Sandbækken et al. (1986)[154]). If longer-duration load increments were used, then the deformation-time curves can be interpreted to determine the end of primary strain for the relevant load increments needed to estimate $\Delta e/e_0$.

NOTE 2 The criteria presented in Table 9 were developed based on results from laboratory tests performed on onshore deposits of marine clays collected from depths below ground surface of 0 m to 25 m and range in properties of 6 % to 43 % for plasticity index, 20 % to 67 % for water content, and 1 to 4 for OCR.

NOTE 3 Relatively high values of $\Delta e/e_0$ can be observed for deepwater clays that are allowed to swell after total stress relief upon sampling and sample handling. This is especially so if gas is dissolved in the pore water in situ and comes out of solution upon sample recovery to deck.

11 Reporting

11.1 Reporting requirements

The scope and extent of reporting shall be defined as part of the project-specific requirements (see 5.3).

An example of reporting format (that suit portable digital format (.pdf) and/or paper print) is given in Table A.8. If the reporting format is not given in project specifications, then the format given in Table A.8 shall be used.

Results of borehole geophysical logging, in situ tests and laboratory tests shall be reported in a suitable digital format according to project specifications. Contour plots, layouts and drawings shall be delivered in a geo-referenced digital format.

11.2 Presentation of field operations and factual data

Presentation of field operations and factual data covers description of equipment and procedures used, detailed list of activities, purpose of the marine soil investigation, marine soil investigation data and results. Presentation of field operations and factual data can include, where applicable:

— plans showing the location of all investigation points, with the coordinate reference system given in the project specifications;

— details confirming measurement uncertainties of the positioning system for each investigation point;

— water depth at each investigation point and the project vertical datum (e.g. LAT or MSL);

— references to or documentation of the methods and procedures applied;

— factual presentation of in situ and laboratory test results including data uncertainties;

— inventory of the laboratory tests performed, both in the field laboratory and in the onshore laboratory, with relevant comments describing the general sample acquisition performance and resulting quality;

— stratigraphic schematization presented in geotechnical logs with a detailed description of all relevant strata;

— evaluation of the data and results, stating the assumptions made and references used for definition of the geotechnical parameters and for quantification of the derived values;

— observations during testing which can impact the results or subsequent interpretation;

— geotechnical design case(s) under consideration for the planned structure (if known and relevant), and other appropriate information about the purpose of the marine soil investigation;

— comparison of the marine soil investigation results with existing experience, in particular previous geophysical and soil investigation data or installation experience obtained locally or in similar soil conditions.

For a given test (borehole geophysical logging, in situ testing or laboratory testing), when no reference standard exists or when the standard procedure has not been used, the applied operating procedure and interpretation method shall be reported together with the results.

Any deviation between the originally specified marine soil investigation scope and the scope of work actually performed should be reported, with the reason for such deviation being clearly documented.

All reported values should be in SI units, with US customary units optionally used as secondary units (if required).

NOTE Project specifications can request a report to be in US customary units only.

Stratigraphic schematization presented in factual geotechnical logs should be interpreted based on available data. Stratigraphic schematization can be improved if the investigated location is on a survey track line of a marine geophysical investigation by providing essential information on the geological setting and variability of soil conditions.

11.3 Data interpretation and soil parameters

The evaluation of geotechnical data and soil parameter values depend on several factors including:

— marine site investigation coverage and ground model;

— quality of data and results;

— spatial variability of material properties within the soil volume of interest;

— geotechnical design situation or analytical framework for which the parameter values can be used as input for structure design.

When correlations are applied to derive soil parameter values (e.g. when determining the undrained shear strength from the CPTU), the correlations used and their applicability to the case under consideration shall be documented. Such correlations can be based on literature in the public domain or on previous relevant experience.

Significant variations in soil conditions can lead to data scatter and uncertainties in the evaluation of geotechnical parameter values. The reasons for such data scatter and uncertainties shall be reported, particularly whether they are due to the natural variability of the soils, applicability of test method, imperfect interpretation methods or other reasons. If some of the data and results are considered as being less representative of the undisturbed conditions than others, these should be identified and should be given less weight in the evaluation of geotechnical parameter values.

When a significant number of data points are available for a given parameter within the bounds of a uniform geotechnical formation, the evaluation of the results can be based on the use of statistical methods to quantify data scatter and uncertainties. Guidance is provided in DNV-RP-C207.

The methods used to establish soil parameter values shall be reported.

Integrated geoscience studies can be performed combining the information gathered in the desk study, marine geophysical investigation and marine soil investigation phases. The principles of a ground model and an integrated geoscience study are given in ISO 19901-10 and SUT OSIG (2022)[98]. Further guidance is also presented in 5.2.2 and A.5.2.2.

Annex A
(informative)

Additional information and guidance

A.1 Guidance on scope

Seabed characterization can require several types of marine site investigations, for example marine soil investigations and marine geophysical investigations including geological and geohazard evaluations. For each project, the types of site investigations required are defined in project specifications. For proper seabed characterization, it is also important that the required investigation equipment and its deployment mode(s) and methods are considered in order to acquire adequate quality soil data to the target depth.

Seabed soils can vary widely, and experience gained at one location is not necessarily applicable at another. The scope of a marine soil investigation for one type of structure is not necessarily adequate for another. Extra caution is therefore necessary when dealing with unconventional soils or unconventional foundation concepts.

Marine soil investigations include both offshore and nearshore soil investigations, which can provide very different challenges.

A.2 Guidance on normative references

No additional guidance is offered.

A.3 Guidance on terms and definitions

This document uses the following terms from the ISO and IEC terminological databases in the context of metrology: accuracy, calibration, combined standard uncertainty, coverage factor, expanded measurement uncertainty, drift, measurand, measurement, measurement uncertainty, measuring interval, measuring interval for calibration, nominal value, relative uncertainty, repeatability, reproducibility, resolution, stability, standard measurement uncertainty, standard uncertainty, uncertainty, verification.

A.4 Guidance on symbols, units and abbreviated terms

No additional guidance is offered.

A.5 Guidance on objectives, planning and requirements

A.5.1 Objectives

No additional guidance is offered.

A.5.2 Planning

A.5.2.1 Sequence of activities

Planning should include the following steps:

a) Understand project drivers;

b) Review of existing data;

c) Identify the preferred timing of data acquisition;

d) Determine vessel and equipment requirements;

e) Determine resources (personnel and logistics) needed to manage and support the work.

The planning and different sequences involved in the definition of a scope of marine site investigations can be summarized as shown in Table A.1.

Table A.1 — Possible planning and sequence for marine site investigations

Phase A		Phase B		Phase C
Desk study		**Marine geophysical investigation**		**Marine soil investigation**
	→		→	
Determination of project phase, type of offshore structure, foundation concept		Seafloor mapping		Recording of drilling parameters and borehole geophysical logging
Evaluation of 3D seismic data (need for re-processing, if any)		Sub-seafloor mapping		In situ testing
Determination of local environmental conditions and site accessibility		Sampling and testing (marine soil investigation)		Sampling
Evaluation of bathymetry and seafloor topography		Assessment of presence of unexploded ordnance		Laboratory testing
Identification of seabed features and obstructions		Assessment of presence of shallow gas		Data integration, interpretation and reporting
Assessment of likely presence of unexploded ordnance		Assessment of active geological features, faults or other geohazards		Clarifying limitations of data and results
Assessment of likely presence of shallow gas		Assessment of soil conditions and variability		Considering any recommendations for additional marine site investigations
Assessment of active geological features, faults or other geohazards		Data integration, interpretation and investigation of geohazards		
Assessment of variability in site and soil conditions		Clarifying limitations of data and results		
Incorporating previous experience in same area or in similar soil conditions		Recommend scope of work for marine soil investigation and any additional marine geophysical investigation		
Assessment of foundation design issues				
Clarifying limitations of data and results				
Recommend scope of work for marine site investigation				

One of the outputs from each phase can be a recommended scope of work for the next phase, with due consideration of the limitations of data and results obtained.

Geohazards can initially be identified through existing geological and historical data and site-specific marine geophysical investigations. Specific data can be required for further evaluation of the geohazards, via methods such as:

— in situ pore pressure measurements to identify gas or gas hydrates, related geohazards, or elevated excess pore pressure that can influence slope stability;

— soil strength to assess seabed resilience to soil mass movement;

— sampling to enable geological description, such as the description and dating of turbidites, of soil in a deposition area of previous debris flows.

In some cases, specialized combinations of marine geophysical investigation and marine soil investigation can be warranted, such as borehole geophysical logging for correlation with the results of marine geophysical investigation, the combination of borehole geophysical logging and sampling for the detection of gas hydrates [Digby et al. (2002)][119] or identification of key soil horizons (e.g. past slope failure planes), or the use of 3D seismic velocities and cone penetration tests to determine the presence of gas hydrates [Sultan et al.(2007)][159].

A.5.2.2 Integrated geoscience studies

Young (2017)[171] gives an overview of the benefits of an integrated study, highlighting that geological and geotechnical engineering analysis and interpretation of site investigation data are fundamentally complementary in an integrated study. The extent and scope of an integrated study should be tailored to address the site complexity in term of spatial variability or possible geohazards and their potential impact on the foundations or anchors for the planned structures.

Conventional 2D or 3D seismic data acquired for oil and gas exploration, preferably HR data or data reprocessed to enhance resolution (see ISO 19901-10), are generally not sufficient. Nevertheless, they are an initial key input for identifying geohazards at a relatively large regional scale, when some geological processes (e.g. slope instabilities up-slope or down-slope, fluid migration processes, tectonic, depositional or erosional features) can affect the planned field development.

For an integrated study for engineering purposes, that has the objective to define a 3D ground model in the top 100 m of sediments, the acquisition of ultra-high resolution seismic data with a vertical resolution in the same order as that of geotechnical data is preferable, i.e. UHR/UUHR data with a target resolution of about 0,5 m (see ISO 19901-10:2021, Table 16).

There are several examples in the literature where geophysical and geotechnical data are integrated, such as those provided in references Campbell et al. (2008)[109], Doyle (1998)[120], Nauroy et al. (1998)[147], Brand et al. (2003)[106], Jeanjean et al. (2003)[131], Moore et al. (2007)[145], Sultan et al. (2007)[159], Thomas (2017)[163], Vardy et al. (2017)[167] and Young (2017)[171].

Shallow seismic reflection data (2D/3D) can allow geotechnical data (essentially 1D) to be extrapolated from (or interpolated between) soil boreholes and test sites. However, the degree of correlation between the geophysical and geotechnical data or the distance over which the stratigraphy from a borehole using geophysical results can be extrapolated depends on the nature, quality and density of the data (both geophysical and geotechnical), as well as on the site geology and the soil characteristics.

Measurements of geophysical parameters should be performed as part of the marine soil investigation, (e.g. shear and compressional wave velocities (v_s and v_p) in geotechnical boreholes or on soil samples in the laboratory). The derivation of v_p can allow the depth-velocity relationship in geotechnical boreholes to be derived, to complement the time-to-depth conversion that can be obtained from seismic data processing and modelling. However, seismic velocities can vary with the frequency bandwidth of the source wavelet, therefore scaling should be considered to compare velocities from a marine geophysical investigation to those obtained from sample testing in the laboratory. Other important aspects that should be considered are sample quality, sample saturation as well as soil anisotropy.

The integration of marine geophysical investigation and marine soil investigation data can be performed in different ways. At a basic level, the objective is to understand how the seismic wavefield propagates through the seabed and to use this information to predict/derive seabed conditions and soil characteristics based on the amplitude, phase, frequency content and attenuation of the marine geophysical data.

The integration of marine geophysical and geotechnical site investigation data by a team of geologists, geophysicists and geotechnical specialists can allow deriving the properties of soil from UHR/UUHR seismic data under the condition of a calibration or validation on available geotechnical data. Some examples are given below:

— The stratigraphy obtained from the seismic interpretation can be used to guide the spatial extrapolation/interpolation of either measured or derived geotechnical properties or their trends (e.g. CPT cone resistance, undrained shear strength) within the identified soil units at the borehole locations, using the stratigraphy as the key element of the ground model (Forsberg et al., 2017)[123].

— The seismic facies and seismic amplitude can be used to identify geological processes and to infer the physical properties of the sediments. For seismic data processed to retain the relative amplitude values, the amplitude is largely controlled by the acoustic impedance (i.e. P-wave velocity times density) contrast across the interface. When geotechnical data are available, the seismic amplitudes can be calibrated against undrained shear strength data or CPT cone resistance data[106].

— Seismic inversion, consisting in transforming a seismic section into a section of acoustic impedance, along with other techniques, can also be used with high quality digital UHR/UUHR seismic data, large volumes of which are now routinely acquired in geophysical site investigations (see ISO 19901-10, Morgan et al., 2012[146] and Vardy et al., 2017[167]). Methods are rapidly developing. For example, Nauroy et al. (1998)[147] and Vardy et al. (2017)[167] describe cases where the P-wave velocity and impedance can be correlated to the in situ CPT cone resistance, suggesting the possibility to extrapolate a cone resistance profile away from a geotechnical borehole via a 2D seismic time section transformed into an impedance section, Carpentier et al (2021)[114] describe the generation of synthetic CPTs using a neural network approach. Such approaches should only be attempted on a site-specific basis and with proper calibration.

A.5.3 Scope of work and development of project specifications

It is important that the marine soil investigation focuses on acquiring sufficient data to meet the design requirements for the offshore structures according to ISO 19900. The types of borehole geophysical logging, in situ testing and laboratory testing should as far as possible be related to chosen or foreseen design methodologies, and to specific parameter values required by such methods.

Interaction between the client and the contractor(s) should occur at an early stage of planning, as well as during the course of a marine soil investigation in order to modify the scope and optimize the outcome of the investigation as knowledge of seafloor and seabed conditions develops. In some cases, the anticipated foundation design requirements for predicted loading can require performance of preliminary foundation check calculations during the course of a marine soil investigation to ensure data adequacy. Identified soil layers that can have an adverse impact on foundation performance can require special attention and characterization.

Examples of such a situation include:

— weak layers that can govern either bearing or sliding failure modes for a gravity base structure;

— interbedded weak and strong layers beneath a jack-up rig foundation which can present a spudcan punch-through risk;

— deep strong layers that can provide high-end bearing resistance for foundation piles.

A reduced marine soil investigation scope can be appropriate where cost-effective and safe design can be achieved by using more conservative geotechnical parameter values, or when there is extensive geotechnical data and experience in the area. For example, explicit tests to achieve specific geotechnical parameter values can be replaced by conservative use of available correlations from simpler tests. In cases where lower quality marine soil investigations are chosen, this should be a deliberate choice by the client.

Considering the range of seafloor and seabed conditions that can be encountered in terms of soil type and strength, the homogeneity of the soil (spatial variability) and how these conditions influence the foundation design, it is not feasible to provide fixed requirements regarding the extent and scope of marine soil investigations. For each particular location and need, the extent and scope of marine soil investigations can be evaluated as a joint effort involving appropriately qualified professionals from a range of disciplines, such as structural and marine engineers, geologists/geophysicists and geotechnical engineers with experience in marine soil investigation and design of offshore structures. ISO 19905-1, ISO 19906, ISSMGE (2005)[94] and other sources such as SUT OSIG (2022)[98], give some examples of typical scopes for marine soil investigations related to various types of facilities, which should be considered as guidelines only.

The typical depth coverage for a marine site investigation can range from a few metres below seafloor for a seabed pipeline to generally less than 100 m below seafloor. However, coverage to 100 m to 200 m can apply for foundations of specific offshore structures, and to possibly 200 m to 400 m if necessary for characterizing geohazards such as shallow water flow sands.

The following factors are typically considered for planning marine site investigations:

a) the type of offshore structures to be designed and the way in which they will interact with the seabed;

b) the needs for determination of the area geology and related geohazards;

c) the prior knowledge of seabed conditions and local geological variability and geohazards including shallow gas, such as the results of desk studies, earlier marine geophysical investigations and marine soil investigations;

d) other considerations, such as local metocean conditions, existing facilities, location of site (e.g. remoteness, possible political instability, medical hazards), unexpected unexploded ordnance, and the possibility of contaminated ground;

e) interaction of investigation equipment with seafloor (see 6.4);

f) regulatory requirements.

Investigation of contaminated ground can require special equipment and procedures. Awareness of special instructions and procedures is important if sampling, storing and testing of contaminated soils is included in the marine soil investigation. When sampling and testing contaminated soil, care should be taken to obtain the correct type of samples, and to seal and store them correctly with respect to the contamination and further analyses.

The variability of the seabed and soil conditions at the site is important for the scope of a marine soil investigation. Investigations are usually targeted at locations of proposed structures, with particular focus on where variations in seabed and soil conditions are anticipated, if of significance to the development, as well as at locations more typical of the overall area of the site.

The choice of vessel, equipment and procedures typically includes the following considerations:

a) Operational requirements, including:

   1) project specific HSE (e.g. possibility of encountering shallow gas);

   2) local metocean conditions, and water depth;

   3) equipment capability to reach target depth;

   4) grouting or not of boreholes after they have been drilled as part of the marine soil investigations (e.g. if the borehole is at the location of a structure, it can be desirable to grout it up, whereas in areas with shallow gas, it can be preferable to leave the borehole open).

b) Quality requirements, including:

   1) accuracy and resolution of control of depth below seafloor;

   2) tolerance of actual investigation locations relative to target locations;

   3) sampling (e.g. whether sharp edged thin wall piston sample tubes are desirable);

   4) need for and testing activities suitable for the anticipated soil conditions;

   5) borehole geophysical logging;

6) seafloor slope or unevenness in terms of deployment of equipment and effects on, for example, depth control.

The planning and the scope of work might need to be modified during the course of a marine site investigation, upon review and evaluation of the results from the investigation as they become available.

When assessing the required depth uncertainty to be followed during a marine soil investigation, it should be considered whether uncertainty of the relation between two data points or the uncertainty of each data point is required, including the level of variability of the quantity being specified. For example, when specifying depth uncertainty, uncertainty of the vertical distance between two points can be more important than the depth uncertainty for each point. Knowledge of the thickness of a soil unit can be required to be known to a greater degree of certainty than its absolute depth. In addition, it should be considered that the depth and thickness of a soil unit can vary across a site, and therefore determining its depth at a given location to a high degree of accuracy can be unnecessary, and possibly misleading, if the variability of depth between locations is substantially greater or unknown.

A.5.4 Health, safety, and environmental requirements for marine operations

A.5.4.1 General

The pre-fieldwork planning phase is critical to the successful completion of a marine soil investigation. The first stage in the process consists of identification of the project objectives, and in particular the factors governing the selection of operating vessel, drilling equipment, sampling equipment and in situ testing equipment.

The second stage of the planning phase is a compilation of a pre-fieldwork documentation package, the principal component of which is a PEP. The PEP is usually prepared by the contractor, with specific input from the client, as required. The PEP should be completed prior to commencing mobilization.

The PEP describes relevant aspects relating to the execution of the marine soil investigation, including HSE, quality assurance, interfaces, management and operations. The PEP contains information of direct relevance to the participants and stakeholders in the marine soil investigation, which can be in the form of instructions, guidelines and reference information.

The scope of the marine soil investigation and the operations are a key component in the PEP. Descriptions typically cover the mobilization phase, through the fieldwork phase, and on to the onshore laboratory testing and data interpretation phases. The complexity of the investigation programme should be considered as well as whether any new or revised equipment or procedures will be employed.

The purpose of the PEP is twofold:

a) to provide assurance to client, contractor and other stakeholders that the proposed scope of marine soil investigation achieves its own objectives (HSE/technical/schedule);

b) to provide a reference for personnel engaged in the marine soil investigation by describing the scope of work, including deliverables and the equipment and procedures to be used, including the strategy to achieve the required positioning accuracy, the responsibilities of each project participant, and the interfaces between participants.

The PEP typically contains the following components:

— a project QP, including QA/QC;

— the objectives and scope of work;

— schedule;

— a marine soil investigation execution plan with particular emphasis on deployment operations;

— a project management and organization plan;

— a project HSE plan (see below) and HSEMS, including an ERP;

— an interface management plan ('bridging document').

An example outline of a PEP for a major marine soil investigation is provided in Table A.2.

A HAZID study is sometimes carried out prior to start of fieldwork, to identify in particular:

— general hazards related to the normal day-to-day operations onboard the investigation vessel, including mooring operations, back-deck operations (e.g. running drill pipes, deploying equipment, working at height and cargo transfer);

— specific hazards related to unusual aspects of the marine soil investigation, such as the use of new or unfamiliar equipment or the use of a specially-mobilized vessel as opposed to a purpose-built permanent soil investigation vessel;

— site-specific hazards that can be encountered during the marine soil investigation, such as seafloor obstructions, shallow gas in soil, UXO, other vessel traffic, and anticipated weather and sea conditions, and their mitigation or avoidance measures as necessary.

After a HAZID study, a HAZOP study is often performed.

The project HSE plan should be completed prior to the start of the fieldwork, and contains:

— a general description of hazards at the site, with JSA;

— generic, project and site-specific HAZID documentation with particular attention to seafloor obstructions or seafloor or sub-seafloor infrastructures;

— an ERP, including provisions for the handling of injured personnel;

— identification of SIMOPS that can impact the marine soil investigation.

Table A.2 — Example outline of a PEP for a major marine soil investigation

Marine soil investigation site name and location Project execution plan (PEP) 1 Introduction 2 Use of project execution plan 3 Purpose of the project 4 Project documentation			
PART A **Scope of work and quality plan (QP)**	**PART B** **Project HSE plan**	**PART C** **Client/contractor HSEMS interface document**	**PART D** **Emergency response plan (ERP)**
Introduction	Introduction and interface statement	Objectives	Emergency response flowchart
Project objectives, scope of work, schedule	HSE Policies and Procedures	Project overview	SIMOPS
Quality policy	Project safety organization and responsibilities	Responsibilities	Communications
Marine soil investigation execution plan	HSE reference documentation	Communications	
Project organization and management structure	Communications	Project scope of work	
Equipment	Emergency	Accident, incident and spill reporting	

Table A.2 *(continued)*

Operating and maintenance procedures	Experience and training	Monitoring, audit and review	
SIMOPS	Working with suppliers and other contractors	Appendices	
Appendices	Hazard management		
	Project health		
	Reporting and investigation procedures		
	Equipment		
	HSE monitoring, audit and review		
	Appendices		

A.5.4.2 Investigation vessel

No additional guidance is offered.

A.5.4.3 Hazardous substances and acoustic noise

No additional guidance is offered.

A.5.4.4 Shallow gas

A.5.4.4.1 Responsibilities

Assessment of shallow gas hazard can be performed by the client, the contractor or both. If performed by the client, the contractor should check that the hazard assessment conclusions are aligned with contractor's own procedures.

With respect to health and safety during operations for marine soil investigation, risk assessment (probability × consequence) for the potential consequences of gas release should be performed by the contractor, with support from the client.

With respect to the environment (e.g. potential gas release into atmosphere, potential disturbance to seabed conditions, equipment left behind in seabed), risk assessment for the potential consequences of gas release is usually performed by the client, with support from the contractor.

A.5.4.4.2 Assessment of shallow gas hazard

There are three possible outcomes of the assessment of shallow gas hazard for a specific location:

a) low probability of encountering gas;

b) intermediate probability of encountering gas;

c) high probability of encountering gas.

A.5.4.4.3 Risk assessment

Typically, the risk assessment should consider:

a) specific activities, such as non-drilling mode deployment, riser/riserless drilling, adding/tripping pipe, sampling/in situ testing and soil sample handling.;

b) potential sources of ignition, entry of shallow gas into confined spaces, etc.;

c) consequences of gas release, such as loss of vessel buoyancy, vessel flooding due to rolling water, jack-up foundation instability, loss of equipment in seabed, fire/explosion, asphyxiation, poisoning (gas toxicity);

d) proportional mitigating measures.

Where applicable, the risk assessment should consider all areas of the vessel (or non-floating platform), i.e. specific areas of potential gas release or accumulation identified, and area classification layouts for the vessel developed. Guidance on hazardous area classification for drilling facilities is given in IEC 61892-7.

Even for a low probability of encountering gas, it is prudent to consider the implementation of residual risk mitigation measures.

Specialized training can be required not only for the operators of marine soil investigation equipment, but also for all offshore personnel, consisting of a course on shallow gas preparedness (operator crew), shallow gas induction (all crew), and regular gas alarm drills (all crew).

In areas with intermediate probability of encountering shallow gas, the following should be considered and implemented, where appropriate:

a) safety meetings with all offshore personnel;

b) appropriate duty watch/chain of command in emergency gas drill;

c) conduct of move-off exercise;

d) no hot work or smoking on deck;

e) suspend all non-essential crane operations;

f) reduce presence of offshore personnel to a minimum in areas of potential gas accumulation;

g) gas detectors (e.g. in the moonpool, in the derrick and near the air intakes of the vessel and engines on deck) and gas alarm;

h) wind and current meters to ensure that the vessel is positioned optimally with respect to the wind and current to minimize accumulation of gas on deck;

NOTE 1 Typically, marine soil investigation operations can continue with horizontal offsets of around 5 % to 10 % of water depth.

i) general look-out or dedicated bridge watch for bubbles surfacing in the vicinity of the vessel;

j) opportunity for increasing bottom hole pressure statically (by having heavy ‘kill mud’ available) or actively (by increasing equivalent circulating density by high capacity pumping system);

k) drilling of a dedicated pilot-hole during daylight hours with a non-return valve in the lower part of the drill string and no running of downhole tools. The drilling of a pilot-hole should be such that it induces swabbing pressures that exceed those expected for the planned marine soil investigation ;

NOTE 2 Drilling of a pilot-hole will not necessarily eliminate the risk of shallow gas. There are examples where shallow gas release took place at short distance from a pilot-hole, even when no shallow gas release was observed for the pilot-hole. The time spent drilling a pilot hole should be representative of the duration of the geotechnical borehole.

l) as a precaution against gas through the drill string, maintain a safety valve on top of the power swivel while using wireline-operated downhole tools;

m) monitor borehole penetration and pump pressure consistently.

In drilling areas with high probability of encountering shallow gas, the following additional precautions should be considered and implemented, where appropriate:

n) TV camera and/or sonar mounted on the seafloor frame or ROV;

o) in situ gas measuring system, in order to ensure early warning before the gas-charged layer is penetrated;

p) pulling the sampling/testing tool out of the soil by lifting drill string while the downhole tool is latched into the BHA with special attention to the swabbing effect when pulling tools out of the soil;

NOTE 3 This practice can reduce or increase risk, depending on equipment configuration.

q) non-return valve in the lower part of the drill string while drilling and pulling pipe.

Depending upon the source, characteristics and severity of the gas hazard, and on the marine soil investigation equipment employed, the following additional equipment can be considered for use in mitigation:

— gas detectors near the air intakes of the vessel and engines on deck, gas alarm and automatic shut-off for unzoned ship's equipment on gas alarm;

— personal portable gas detectors;

— breathing apparatus;

— intrinsically safe communications system;

— isolated moonpool chamber (with fresh air duct or HVAC system);

— gas diverter on top drive unit;

— shear rams;

NOTE 4 This measure can reduce or increase risk, depending on equipment configuration.

— gas-tight valve in seafloor to tank for drilling mud;

— non-return valve in BHA;

— zoned drill floor equipment.

A.5.5 Other requirements

A.5.5.1 Operational requirements

No additional guidance is offered.

A.5.5.2 Quality requirements

No additional guidance is offered.

A.5.5.3 Specific considerations for unconventional soils

Examples of unconventional soils and their characteristics presenting potential difficulties are provided in Table A.3.

Table A.3 — Examples of unconventional soils and their characteristics presenting potential difficulties

Description	Potential difficulties
Artificial soil / contaminated soil / made ground	Contamination; human danger
Ash deposit	Strength, compressibility and permeability
Calcareous clay and silt	Cemented layers; soluble
Calcareous sand	Low strength and stiffness upon application of stress; cemented layers; soluble
Claystone and shale	Swelling upon reduction in stress
Coal	Flammable; mining activities
Dispersive soil	Internal erosion upon flow of water through soil; formation of cavities
Evaporitic rock	Soluble; mining activities
Ferrous soil	Cemented layers
Fissured clay	Low strength; high permeability
Gassy soil	High compressibility; low strength; explosive; flammable; toxic
Glauconite sand	Weak particles; 'clay' behaviour; low permeability
Gypsum and anhydrite	Volume change
Loess	Instability of soil structure upon disturbance
Mica sand	Instability of soil structure upon disturbance
Ooze	Low strength, instability of soil structure upon disturbance
Organic soil	High compressibility; susceptible to chemical change
Peat	High compressibility; susceptible to chemical change
Permafrost	Influence of temperature on soil structure, soil loss and gas presence. Ice content is also an important factor.
Residual soil	Strength, compressibility and permeability strongly depend on soil structure; non-homogeneous differential weathering; occurrence of boulders
Saline alkali soil	Volume change; aggressive towards concrete and steel
Sensitive clay (or quick clay)	Loss of strength upon application of shear strain
Shrinking / swelling clay and silt (or active clay)	Loss of strength and stiffness upon increase in water content; change in volume upon change in water content
Strong rock or better	Rock burst upon reduction in stress; rapid wear of drill bit
Under-consolidated clay and silt	Ongoing compression under self-weight
Varved clay (or laminated or stratified clay)	Anisotropy in strength, stiffness and permeability
Volcanic soil	Instability of soil structure upon disturbance; low density

A.6 Guidance on deployment of investigation equipment

A.6.1 Non-drilling mode deployment

No further guidance is provided.

A.6.2 Drilling mode deployment

No further guidance is provided.

A.6.3 Uncertainty of vertical depth measurements

A.6.3.1 General

This subclause provides information on the various deployment systems currently in use and the main factors affecting the uncertainty of vertical depth. The information provided herein is not exhaustive. It is essential that a robust assessment of the key factors affecting vertical depth uncertainty is made and reported.

Vertical depth measurement is largely influenced by the mode of operation of the investigation equipment. This subclause summarizes the key issues that affect the uncertainty of vertical depth measurements for alternative types of equipment and the common parameters should be considered in the calculation of vertical depth uncertainty.

A.6.3.2 Factors affecting the uncertainty of vertical depth measurements

A.6.3.2.1 Vessel drilling systems

The following factors can influence the uncertainty of data point depth measurements for vessel drilling systems, where applicable:

a) the performance of a vessel's heave-compensation system in prevailing sea conditions;

NOTE 1 An indication of the limiting sea state for which an estimated depth uncertainty is likely to be exceeded should be reported.

b) the type of heave-compensation reference system used;

NOTE 2 'Hard-tie' systems that utilize a template founded on the seafloor (as a reference against vessel heave) generally provide better drill-string stability than 'non-hard-tie' heave-compensation systems.

c) the water depth, and consequently the length of drill string deployed;

NOTE 3 Longer drill strings are more susceptible to lateral displacements associated with sea currents, and drill string 'whipping' [see Figure A.1 b)].

d) the weight and elasticity of the drill pipe;

NOTE 4 Heavier drill strings require stiffer heave-compensation systems. Stiffer systems are less sensitive to changes in seabed strength, and hence offer less feedback to the driller, particularly in weak soil layers.

e) the uncertainty of the seafloor tagging exercise undertaken prior to the commencement of drilling;

NOTE 5 This is a critical measurement that is used to measure the water depth and account for current vessel draught, prior to commencement of drilling.

f) the uncertainty of the echo sounder, GNSS (or similar) equipment used to monitor tidal fluctuations;

NOTE 6 This is particularly important in regions that have high tidal fluctuations.

g) the measurement uncertainty of drill pipe and bottom-hole assembly lengths;

h) the presence/absence of a seafloor template, the ability of the seafloor template to land, level and fully stabilise itself on the seafloor, and the capacity to monitor its height relative to the seafloor, including any settlement of the template into the seabed;

i) the capacity of the vessel to maintain its horizontal position in prevailing sea conditions;

NOTE 7 Spatial deviation of the drill floor centre relative to the borehole centre will affect the estimated length of drill string downhole.

j) push-in penetration control of a deployed tool, relative to the bottom-hole assembly.

A.6.3.2.2 Seafloor-founded systems

Seafloor-founded systems are initially landed and stabilised on the seafloor, prior to acquiring data or prior to sampling. This includes seafloor drilling systems (see 6.2.3) and some seafloor-founded, non-drilling mode systems (6.1).

The following factors can affect the uncertainty of depth measurements for such systems, where applicable:

— the capacity of the system to land, level and fully stabilise itself on the seafloor prior to and during the course of a seabed investigation;

— the method of isolating the seafloor-founded system from surface vessel movements;

NOTE 1 Some systems are landed on the seafloor and a catenary is formed in the umbilical prior to commencing investigation activities. Provided sufficient umbilical is paid out to accommodate vessel station-keeping requirements, this allows the seafloor-founded system to be fully isolated from vessel movements.

NOTE 2 Alternatively, some systems are lowered to the seafloor on a constant-tension winch. Provided the surface environmental conditions remain within the dynamic operating capabilities of the winch, such systems can also provide effective isolation from surface vessel movements.

— the uncertainty of the vertical (drill-head, push rod) position transducer;

— the capacity of the system to measure and monitor the height of the equipment relative to the seafloor, including any settlement of into the seabed [see Figure A.1 a)];

— the length, weight and elasticity of the drill string or push-rod assembly used to push the tool into the seabed;

— the verticality of the tool within the borehole.

A.6.3.2.3 Non-seafloor-founded systems

Investigation equipment not founded on the seafloor prior to the commencement of data acquisition (e.g. a free fall penetrometer, gravity core sampler and piston core sampler without fixed reference to seafloor) can be deployed in free-fall mode or can remain partially to fully suspended on a non-heave-compensated winch wire during the seabed penetration phase. The data acquisition process is typically of short duration only; thereafter the investigation tool is immediately recovered to the vessel deck.

Control of depth uncertainty can be poor for such systems, as the effects of vessel heave, in conjunction with the simplicity of the tool design, can impart uncontrolled, variable movement to the tool during the data acquisition process.

The following factors can affect the depth measurements for these types of systems, where applicable:

— vessel heave during interception and penetration of the tool into the seabed;

— poor estimation of the penetration of the tool into the seabed;

NOTE These types of equipment are generally not equipped with a seabed-penetration measurement device.

— the extent of penetration of the trigger plate into the seabed, prior to tool release;

— the weight of the lift cable and its elasticity;

— the uncertainty of the cable-payout meter;

— the level of feedback provided to the winch operator when the system trips;

— the verticality of the tool during seabed penetration;

— the horizontal offset of the tool relative to the deployment point on the vessel.

Estimation of data point depth uncertainty for such systems is generally difficult. For this reason, the depth uncertainty class can be assumed to correspond to depth uncertainty class Z5 (>2 m as given in Table 1) in the absence of further documentation.

The limiting surface environmental conditions, beyond which an agreed depth uncertainty class cannot be achieved, should be clarified.

A.6.3.3 Depth uncertainty classes

A.6.3.3.1 Estimation of data point depth uncertainty

Data acquired from a marine soil investigation programme are usually presented versus vertical depth below seafloor, where depth is normally quoted as a downwards positive value below seafloor. All depth measurements quoted will have an uncertainty associated with their measurement. As discussed in 6.3, the uncertainty of data point depth measurements is an important parameter for the analysis of geotechnical results.

Three alternative approaches to depth measurement are possible:

a) measurements are made relative to a reference point at or close to the seafloor [see Figure A.1 a)];

b) measurements are made relative to a reference point at or close to sea level [see Figure A.1 b)]; or

c) a combination of the above [see Figure A.1 b)].

Values for h_s, h_d, h_{sf} and d_w in Figures A.1 a) and b) are either estimated or a direct physical measurement is undertaken. The basis for estimation can include experience or rule of thumb, visual observation against some scale reference, theoretical calculation or physical modelling.

Basic principles for physical measurement typically include one or more of the following:

— tape measure;

— mechanical distance sensor;

— optical distance sensor;

— acoustic distance sensor;

— pressure sensor;

— fluid flow sensor;

— accelerometer;

— inclinometer.

Supplementary and complementary measurements can improve the confidence in the principal measurement. For example, for vessel-drilling systems the direct measurement of the seawater temperature profile can improve uncertainty of values obtained by marine acoustic distance measurement (echo sounder or altimeter).

Each estimate or measurement will have an error. Estimates of these errors can be arithmetically added to obtain an overall estimate of the uncertainty of depth measurements. For complex systems, this approach can be conservative, and more rigorous estimates of depth uncertainty can be obtained via the separation of random and systematic errors, and the application of probabilistic principles.

A systematic error affects the mean value of a measurement and, as opposed to a random error, is always in one direction. Calibration error can be an example of a systematic error. Random errors result in scatter if the measurement readings are plotted on an *x*-*y* plot and affect the variance of a measurement.

Some key parameters for assessing the uncertainty of data point depth measurements are provided in A.6.3.3.2 to A.6.3.3.6.

A.6.3.3.2 Identification of the seafloor–water interface

The seafloor is commonly used as a reference datum for depth measurements. The following factors should be considered in the estimation of seafloor level:

— Seafloor elevation can change due to erosion or deposition events during deployment of investigation equipment and, more generally, with time.

— Deployed investigation equipment can partially displace very soft upper seabed materials during approach and touchdown and operation.

— The soil-water interface can be difficult to identify. Seafloor sensors, which identify the seafloor interface via measurement of upper seabed density or soil strength, have different threshold settings. Moreover, mechanical seafloor detection sensors do not necessarily detect a near-fluid upper seabed.

— An acoustic distance sensor typically records the highest point within its beam, which does not necessarily coincide with the point of investigation for an undulating or sloping seafloor.

A.6.3.3.3 Height of reference point above seafloor, h_{sf}

A seafloor frame can penetrate into the seabed upon touchdown. Measurement of seabed penetration is important for systems that measure depth relative to the seafloor frame itself.

A seafloor frame can vertically displace and/or tilt as a result of:

— self-weight of the seafloor frame;

— forces generated during investigation activities;

— forces applied by guide wires;

— heave-compensated umbilical tethers;

— eccentric loading of footings on seafloor slopes.

A.6.3.3.4 Depth below seafloor, z

Estimation of tool-point depth is usually made via measurement of the length of drill string or push rods deployed downhole (L_{string}, see Figure A.1), in relation to a fixed datum point located either at the seafloor (seafloor-founded systems) or at the drill floor (vessel-drilling systems).

The following factors can affect the uncertainty of depth measurements:

a) errors in estimated tool and drill string/push-rod measurements, including:

   — measurement of the actual length of tools;

   — elastic shortening or lengthening of tools under varying thrust/tension loading;

   — ‘out-of-straightness’ of a drill string within a water column (e.g. where slender drill strings are subjected to drag forces associated with sea currents);

   — non-verticality of a drill string or push rods within the seabed;

   — thermal shortening or lengthening of drill string/push rods;

b) mechanical errors within thrust machines (e.g. the slippage of gripping mechanisms used to either clamp and/or thrust tools into the seabed);

c) electrical instrumentation errors, including:

— hysteresis and nonlinearity of displacement transducers;

— measurement errors within inclinometers used to estimate the verticality of deployed tools.

Where sampling is undertaken, the assignment of sample loss/extension within a sampling run can be based on engineering judgement. This can reduce uncertainty for data-point depth measurements compared to a default assumption for sample loss. Further guidance on the issues of sample loss/extension, and how this factor can affect the estimated depth range over which a sample has been acquired, is provided in 9.4.

A.6.3.3.5 Water depth, d_w

The measurement of water depth is particularly important for vessel-drilling systems. Water depth variations occur due to natural factors such as tides, currents, wind and barometric pressure. The influence of water depth estimates on data point depth uncertainty can include:

— variation in vessel draught with time, arising due to variations in seawater salinity and the quantity of fuel/water retained onboard, for example; the position of a fixed echo sounder below the water surface is usually directly affected by vessel draught.

— errors associated with the sounding of the seafloor at the commencement of seafloor operations;

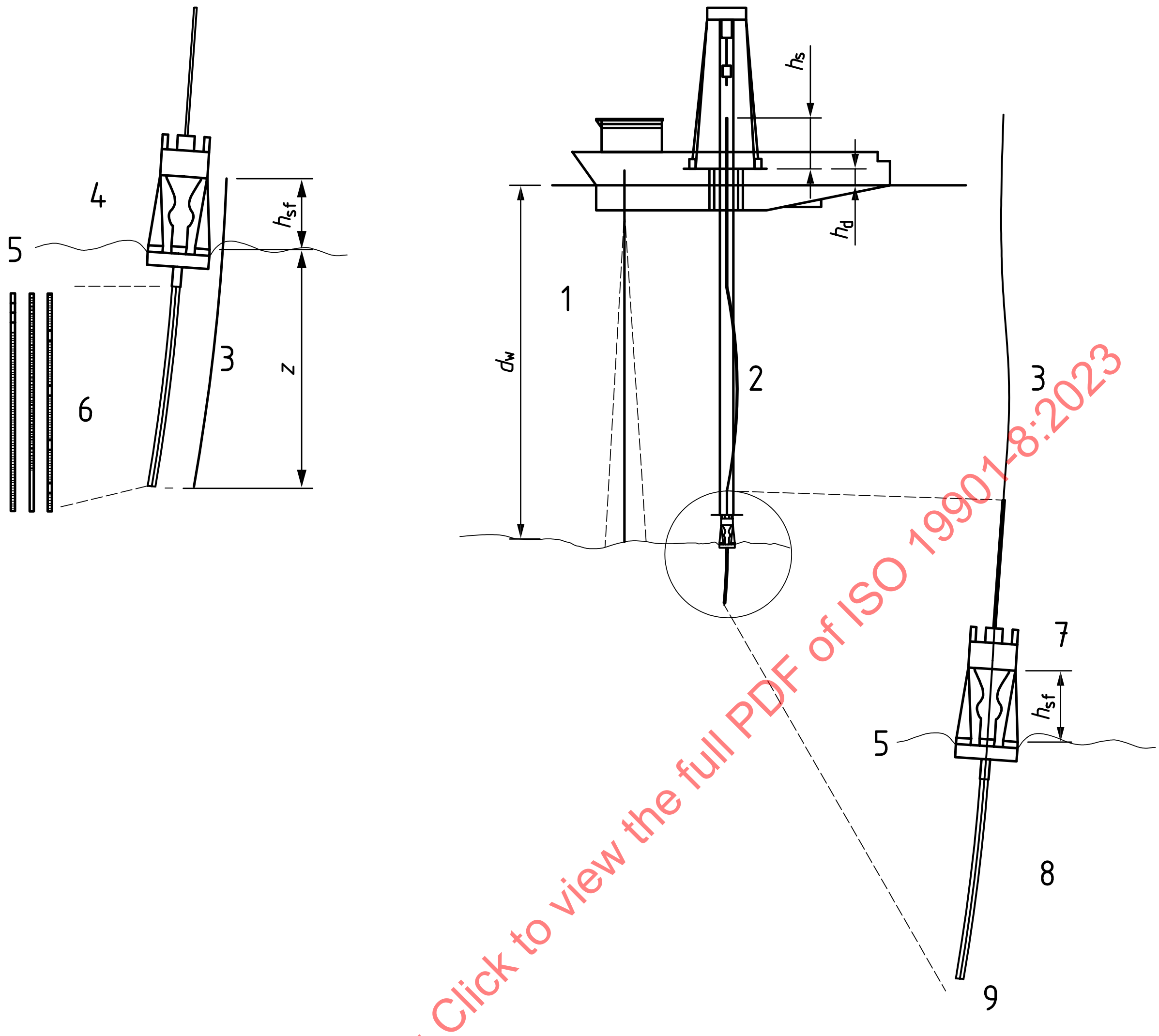
NOTE A direct sounding of water depth by drill string can be achieved by fixing the drill pipe in a seafloor frame at the seafloor. The straightness and verticality of the drill pipe depend on factors, such as weight on hook in drill derrick, seawater current and position of the seafloor frame relative to the drill derrick. An underwater positioning system mounted on the seafloor frame can provide information about the relative position of the seafloor frame. Length variations in drill pipe are discussed in A.6.3.2.1.

— variations in barometric pressure, seawater salinity, turbidity and temperature, which can affect water depth derived from pressure sensors at the seafloor.

A.6.3.3.6 Height of reference points above sea level, h_s and h_d

For vessel-drilling systems, uncertainties associated with height of reference points above sea level should be considered. Particularly the height of reference points above sea level, h_s and h_d [see Figure A.1 b)] can vary due to a change in vessel draught.

Height of a tool joint above the drill deck can be estimated using tape measurements and/or visual observation. Observational measurement can be particularly difficult where operations are being undertaken during conditions of significant vessel heave.



a) Seafloor-based measurement, with reference point close to seafloor **b) Vessel and seafloor-based measurement, with reference point close to sea level**

Key

1 echo sounder or alternative water depth measure
2 drill string out-of-straightness
3 L_{string} (length of drill string or push rods deployed downhole)
4 tilt and/or sinking of seafloor frame into seabed
5 seafloor
6 effect of sample loss on data-point depth uncertainty
7 tilt and/or sinking of seafloor template into seabed
8 curvature of drill string
9 tilt of tool below drill string

Note that h_s and h_d are height of reference points above sea level. h_{sf} is height of reference point above seafloor.

Figure A.1 — Examples of factors affecting measured depth

A.6.4 Horizontal positioning

No further guidance is provided.

A.6.5 Interaction of investigation equipment with the upper seabed

A.6.5.1 Minimization of seafloor disturbance effects

A.6.5.1.1 General

Most types of marine soil investigation systems interact with the seafloor in one way or another. Vessel-drilling systems with heave-compensation systems are reliant on seabed templates to provide a fixed reference against vessel heave, while seafloor-founded systems are directly founded on the seafloor prior to commencement of data acquisition.

Vessel-drilling systems normally deploy reference frames or templates to the seafloor prior to deployment of the drill string. These devices provide reaction thrust as tools are pushed into the seabed, to enable hole re-entry where required, and to provide a vertical datum against which the vessel's heave-compensation system is referenced. Seafloor-founded rigs are also located on the seafloor throughout the course of seabed investigations; these machines can impart variable loading to the seafloor as operations progress.

The landing of equipment on the seafloor prior to the start of the investigation can cause disturbance to the seabed. The transfer of equipment weight from the vessel to the seafloor can lead to artificial surcharging of the upper seabed. This can influence sample quality and *in situ*-measured test results. Reporting should include an assessment of the interaction of the deployed investigation equipment with the seafloor, and whether this can have adversely affected the measured upper seabed properties during the course of the soil investigation. This can be particularly important where equipment is landed and operated on very soft seabed.

A.6.5.1.2 Measures to mitigate near-seafloor soil disturbance

While for some cases it is possible to apply approximate corrections to test data for seafloor disturbance effects, artificial seafloor disturbance should be minimized wherever possible. Disturbance can be minimized via the following techniques:

— careful control of the rig during initial touchdown with the seafloor, to avoid rapid and uncontrolled impact;

  NOTE Careful control can include landing of equipment at a time that coincides with a period of reduced vessel heave (for example between the passage of wave sets).

— use of adequately sized 'outrigger' footings to transfer weight of the rig to the seafloor, well clear of the sampling/testing zone, at the point of initial rig landing on the seafloor;

— minimization of the 'on-bottom' weight of the rig and maximization of the bearing area of the footings to reduce applied bearing pressure;

— providing a means to adjust the rig's base elevation, as required where seafloor contact is made, during seafloor operations;

— use of foundation 'skirts' on a footing system, particularly where operating on very soft seabed.

A.6.5.2 Operations on steep seafloor gradients

A.6.5.2.1 General

Marine soil investigations on steep and/or uneven seafloors can be required. Examples include:

— pipeline route investigations that traverse from deepwater to continental shelves;

— seabed investigations in areas of previous slope failures;

— investigations in the vicinity of iceberg gouges;

— investigations on rocky seafloors;

— operations within the vicinity of sand waves.

Operations on steep slopes present additional challenges to the safe operation of seabed investigation equipment. Information on seafloor slopes is normally obtained from the interpretation of marine geophysical investigations or 3D seismic data. This information should be made available during the early stages of equipment selection, if steep slopes can be encountered.

The capabilities (and where appropriate the limitations) of the proposed geotechnical investigation equipment to effectively operate on the expected seafloor slopes are the responsibility of the contractor.

Based on the technical description provided by the contractor, the client should familiarize with the capabilities of the proposed investigation equipment, and the potential impacts on the quality of the acquired data. Special deployment methods, and/or specialist investigation equipment, can be selected to ensure accurate data are acquired in a safe manner.

A.6.5.2.2 Factors affecting equipment performance on steep slopes

The following factors should be considered in assessing equipment performance on steep slopes:

a) the expected strength of the upper seabed at the proposed investigation sites, and whether the applied surcharge and operation of the equipment, resulting in vibrations and additional torsional or vertical loading, can lead to seabed instability;

b) whether the ‘out-of-levelness’ of the template will impede the passage and operation of the drill string;

c) the capacity to measure the height of the drill/push centre above the seafloor at the centre of the machine;

d) the local variability of the slope, and whether this is likely to affect the capacity of the system to land and remain stable during drilling operations;

e) the capacity for equipment to operate on non-uniform (e.g. rocky) seafloor slopes.

A large proportion of equipment currently available for marine soil investigations can safely operate on seafloor slopes of up to 5°. Wherever seafloor slopes are identified at an investigation site, the practical operating capabilities of the selected equipment should be taken into account.

A.7 Guidance on drilling and logging

A.7.1 General

No additional guidance is offered.

A.7.2 Project-specific drilling requirements

No additional guidance is offered.

A.7.3 Drilling objectives and selection of drilling equipment and procedures

A.7.3.1 General

A variety of drilling methods and equipment are available. This document primarily considers rotary drilling techniques.

In rotary drilling, the rate of penetration (ROP) depends on a combination of the characteristics of the bit, the pressure and volume of flow of drilling fluid (mud pressure and mud flow), the normal force between the bit and the bottom of the borehole (weight on bit), and the characteristics of the material being cut. Depending on the objectives, the driller adjusts the combination of weight on bit, drilling fluid pressure and flow, and drilling fluid characteristics in order to advance the borehole.

When drilling from a floating vessel, cyclical variations in weight on bit (and hence soil disturbance) can be induced by vertical motions at the drill floor. These motions are attenuated by employing a heave-compensation system. Appropriate equipment should be identified for the defined drilling objectives. Where optimal data quality is an objective, the hard-tie heave-compensation system offers a high performance option, particularly in softer soils.

Appropriate drilling equipment and methods should be identified for the defined drilling objectives. Within the two main modes of deployment (i.e. vessel-drilling mode and seafloor-drilling mode), the typical types of rotary drilling are:

— 'open-hole' or 'core' drilling;

— 'uncased' or 'cased' drilling;

— 'riserless' or 'riser' drilling.

A.7.3.2 Open-hole versus core drilling

In rotary open-hole drilling, sampling and in situ testing are performed on undisturbed material ahead of the bit by pushing (and other means such as percussion) specialized wireline tools through the open centre of the bit.

In rotary core drilling, rotary core samples are recovered either as cores typically 1,5 m to 3 m long, recovered by a drop-in wireline coring system, or as long continuous lengths of core typically 6 m or more, recovered by pulling the complete drill string out of the borehole to recover the core barrel (rotary core sampler). However, in the latter case, a re-entry system is needed in order to re-enter the existing borehole to advance to the next core interval.

The drilling technique adopted is largely governed by the expected soil conditions, the requirement for sample quality, and required in situ testing and borehole geophysical logging. Open-hole drilling with sampling and testing by downhole tools is generally appropriate for uncemented formations, and for some weakly cemented soils or very weak rock formations (e.g. highly weathered chalk). Rotary core drilling is generally appropriate for cemented soils or rock formations and can also be a good alternative in very stiff boulder clays, especially if recovery is more important than sample quality.

In some cases, a combination of the two techniques can be employed, with three different coring systems and procedures, given below, that can be adopted.

a) Pull out the open-hole drill string completely and run in a rotary coring drill string. This approach requires additional pipe handling, but maximizes core sample diameter and ensures that the most appropriate drill bit is always used for the formation encountered.

b) Pull the open-hole drill string back a short distance from the bottom of hole, then run a coring drill string through the centre of the open-hole string (termed 'piggy-back coring'). This approach requires some additional pipe handling, possibly an additional top drive unit, and the sample diameter is reduced, but borehole integrity is ensured and the most appropriate drill bit for the formation encountered can be used. This mode of operation requires a seafloor frame to be clamped onto the drill string in order to prevent the open-hole drill string from heaving within the borehole due to the motion of the vessel.

c) Lower a wireline rotary core sampler through the drill string to latch into an open-hole BHA. This approach saves pulling or running any additional drill string but requires continued use of the open-hole drill bit, which can be less appropriate for the seabed to be sampled and can result in poorer quality sample and/or reduced recovery.

A.7.3.3 Uncased versus cased drilling

Most marine soil investigations are performed using equipment in uncased, open-hole rotary drilling mode, where a single tubular (the drill string) runs from the drilling platform.

At sites where borehole stability can be an issue (such as excess pore pressure regimes or very deep boreholes), or where project-specific objectives demand (e.g. ‘piggy-back coring’), it can be necessary to install one or more casing strings prior to completing the borehole. Casings can be either specialized casing tubulars run separately, or drill pipes doubling as casing once a certain depth (or other criterion) has been achieved. Multiple concentric casing strings, reducing in diameter as the borehole advances, can be employed and specialized handling equipment and procedures can be required.

A.7.3.4 Riserless versus riser drilling

Most marine soil investigations are performed in riserless drilling mode, where drilling fluids and cuttings flow between the annulus of the borehole and the outside diameter of the drill pipe and end either at the top of the borehole (i.e. at seafloor) or at the top of the casing string. With riserless drilling (also termed ‘total loss drilling’ or ‘drilling with fluid returns to seafloor’), all drilling fluid and cuttings exit the borehole in a plume which is either dispersed by sea-bottom currents, or from which cuttings settle out of suspension to form a mound in the vicinity of the borehole.

With riser drilling, a tubular casing extends from the borehole back up to the floating vessel or platform. This allows the recovery of drilling fluid and cuttings for recirculation, for retention of the fluids and cuttings (e.g. in areas where zero discharge-to-sea is required) or for sampling of the cuttings. Riser drilling requires specialized handling and processing equipment and procedures (see ISO 22475-1). In particular, care is needed to ensure that the elevated hydraulic pressure exerted on the borehole walls by the hydraulic head of drilling fluids between the water surface and the riser top does not lead to hydraulic fracture of the formation, which can lead to loss of drilling fluid into the formation and can compromise borehole stability or sample quality.

A.7.3.5 Drilling equipment applicability considerations

In order to assess the applicability of a particular drilling spread of equipment for a given set of project-specific requirements, details of the equipment should be reported, including where applicable:

— operating platform description (i.e. floating or fixed platform, moonpool or over-side deployment, submersible drill rig) with nominal operating limits (i.e. sea/tide/current/wind);

— drilling system capacity, with drill mast/derrick height, weight capacity under the power swivel (i.e. maximum mass of drill string including drill collars and BHA), and details of draw-works heave-compensation system (type, maximum capacity, full-stroke length and usable-stroke length) if in floating vessel mode;

— specification of drill pipes and drill string handling system, with maximum length of drill string that can be handled and maximum pipe stand length;

— description of drill bits carried onboard, and rotary coring system (if any);

— details of the seafloor frame, and of its heave-compensation system (type, maximum capacity, full-stroke length, usable-stroke length);

— mud system details, with mud pump(s), mixing capacity, number and volume of mud tanks, maximum pumping rate;

— drilling parameter logging system (i.e. automated or manual, parameters logged and system limitations);

— personnel levels required on the drill floor, with location of controls for drilling and equipment deployed within the drill string, pipe and wireline tool handling systems.

Table A.4 offers guidance on drilling equipment characteristics relevant to particular project requirements. It can be used to help identify appropriate or inappropriate equipment.

Table A.4 — Drilling equipment selection considerations

Priority	Project-specific requirements	Principal drilling equipment selection considerations
1	Water depth and sea conditions	Seafloor drilling/vessel drilling/availability/hook capacity/power/heave compensation
2	Soil conditions	Open-hole/rotary coring/uncased/cased/riserless/riser/power
3	Borehole geophysical logging, in situ testing and sampling	Open-hole (or combination)/seafloor reaction frame/heave compensation
4	Borehole depth	Uncased/cased/riserless/riser/power
5	Drilling hazards	Uncased/cased/riserless/riser/pilot hole/specialized equipment
6	HSE, sustainability and extreme working environment	Specialized equipment (minimized manual drill floor interventions), mud discharge, carbon footprint
7	Environmental discharge	Riserless/riser
Modifier	Optimized data quality	Accurate heave-compensation/seafloor reaction frame
Modifier	Optimized data recovery	Working height (maximum length of downhole tool that can be handled)
Modifier	Optimized rate of progress	Power/mud flow

A.7.3.6 Drill bit selection

The selection of drill bit should be made based on the expected soil conditions, required drilling, logging, sampling and/or in situ testing techniques, and whether optimal sample quality or maximum rate of penetration (ROP) is required.

Where sampling or testing is required, an open-centred bit (‘core bit’) is used in open-hole drilling mode. For drilling without sampling or testing, a closed-centred drilling bit can offer greater ROP. Also available are drop-in ‘centre’ bits which latch into the BHA to convert an open-centred bit into a closed-centred bit, and can subsequently be recovered by wireline overshot.

For open-hole drilling in uncemented soils, a drag bit is likely to be the preferred solution. The drag bit is particularly appropriate where it is expected to encounter clay strata (which can have a tendency to ‘ball’ and block the discharge ports of other bit designs, which can lead to bit damage and/or significantly reduced ROP). For other ground conditions (principally rock), ISO 22475-1 provides additional guidance regarding bit selection.

The following factors should be considered for drill bit selection:

a) range of bits for the possible soil conditions, drilling problems and hazards that could be encountered;

b) criteria for change of drill bit;

c) positioning of drilling fluid discharge ports and bit geometry (i.e. the bit should be balanced to minimize the tendency of the borehole to deviate from the axis).

A.7.3.7 Drilling fluid

The function of the drilling fluid is twofold: to carry drill cuttings away from the cutting face, and to prevent the drill bit from overheating. However, depending upon its specific characteristics, the fluid can also serve a number of other functions:

— borehole advancement, i.e. cutting soil ahead of the bit by hydraulic action;

— borehole stability, i.e. formation of a stable 'cake' around the borehole thereby helping prevent cave-in of the borehole walls, particularly in loose non-cohesive or over-pressured formations;

— prevention of drill cuttings from settling back to the bottom of the borehole while pumping is stopped, for example when running a sampling or in situ testing tool;

— control of hydrostatic pressure in the borehole.

Drilling fluids are typically based on seawater (or fresh water), and in some cases water alone is sufficient, provided the mud system is able to deliver adequate flow rate and pressure. When defining the drilling fluid to be used, consideration should be given to the potential for adverse chemical interactions of the drilling fluid (including seawater) on the soil (e.g. low salinity seawater with high salinity soil).

If the drilling fluid is required to perform additional functions, then one or more additives can be included to give the necessary characteristics (e.g. the addition of barite for increasing hydrostatic pressure in the borehole). Additives can be supplied in liquid or powder form, and specific mixing procedures should be followed (with due regard to HSE considerations). It can be necessary to periodically test samples of the mixed fluid to ensure that the desired consistency and viscosity characteristics are maintained across multiple batches.

For complex borehole constructions, or in difficult soil conditions, advice should be sought from experts regarding design of an appropriate drilling fluid programme.

A.7.4 Drilling operations plan

All drilling equipment used during the investigation should have documentation to include as a minimum:

— Operational procedure;

— schematic drawings showing overall dimensions and required deck space;

— Safe working limits;

— Ancillary equipment;

— Calibration documentation for measuring devices;

— Manufacturer/suppliers specifications;

— Details of emergency shut off switches

Prior to drilling, the water depth should be determined by at least two independent methods which may include the following:

— Hull mounted echo sounder

— Depth gauge mounted on reaction frame or seabed drilling system

— Drill string length

— Acoustic beacon on seabed frame

— Wireline tool

A.7.5 Recording of drilling parameters

No additional guidance is offered.

A.7.6 Borehole geophysical logging

A.7.6.1 General

This section covers wireline borehole geophysical logging tools, examples of which include:

— Caliper tool: measures borehole diameter by a mechanical caliper providing information on borehole rugosity and general borehole condition (e.g. borehole stability, swelling, caving). Open hole logging only;

— Orientation and temperature tool: measures borehole fluid temperature and borehole deviation and azimuth. Open hole, in-casing or in-pipe logging. Borehole azimuth only available in open hole logging;

— Natural gamma radiation tool: measures gamma radiation naturally emitted by the formation, providing inferred information on soil and rock type which can be used for correlating data over several runs or closely spaced boreholes. The tool has no radioactive source. Open hole, in-casing or in-pipe logging;

— Spectral gamma radiation tool: measures natural gamma radiation within the gamma ray spectrum, quantitatively inferring the relative contents of 40K, 232Th and 238U in the formation, allowing mineral identification and improved lithology interpretation. The tool has no radioactive source. Open hole, in-casing or in-pipe logging;

— Resistivity tool: measures electrical resistivity of the soil, providing interpretative information on soil and rock type, differences in unit weight (porosity) of the soil and nature of pore fluid in the soil. Open hole logging only;

— Magnetic resonance tool: measures the response of hydrogen spins to a magnetic field perturbation, providing interpretative information on soil parameters such as water content, porosity, pore space distribution and permeability. Open hole, or in-casing (plastic) logging only;

— Sonic tool: measures acoustic waveforms for deriving wave velocity within the formation using a built-in acoustic source (monopole, dipole or quadrupole). The sonic data can provide indication on soil type and differences in soil character. Open hole logging only;

— Vertical seismic profile: measures acoustic waveforms for deriving discontinuous (requiring a static position of the tool) primary (P-) wave interval velocities and, where applicable shear (S-) wave interval velocities. The vertical seismic profiler utilizes a seismo-acoustic source away from the borehole, suspended in the water column (e.g. airgun) or mounted on a seafloor template (e.g. impact hammer). Open hole logging only;

— P and S suspension logger: measures acoustic waveforms for deriving discontinuous (requiring a static position of the tool) primary (P-) wave and shear (S-) wave interval velocities in the formation. The P and S suspension logger utilizes a built-in acoustic dipole source. Open hole logging only;

— Acoustic televiewer: measures travel time and amplitude of high resolution ultrasonic acoustic waves which are emitted and received by the tool after reflecting off the borehole wall. The acoustic televiewer also measures tool inclination and azimuth. The acoustic televiewer provides an oriented, 360° continuous image of the borehole wall which is recorded as both an amplitude image and a travel-time image. Further analysis of both images can provide information on orientation of discontinuity planes within the formation. Open hole logging only;

— Optical televiewer: generates a continuous oriented 360° image of the borehole wall using an optical imaging system. The optical televiewer also measures tool inclination. Further analysis of the image can provide information on orientation of discontinuity planes within the formation. Clear borehole fluid and open hole logging only.

Other types of logs used in offshore activities exist, including density and photoelectric factor logs, neutron logs, dipmeter, imaging logs, self-potential logs, spectral noise logging (such logs are often used in e.g. ocean drilling projects).

Test apparatus includes the following, where applicable:

— Borehole drilling equipment, including borehole fluid (drilling mud);

— Casing pipe (e.g. threaded plastic casing);

— Logging winch equipped with wireline cable, providing mechanical support and electrical communication for downhole logging tools, and a depth and tension measuring device;

— Borehole geophysical logging tools, connected (in sequence) with the wireline cable, interface panel and acquisition/processing computer;

— External acoustic source (e.g. airgun);

— Surface reference electrode.

Borehole geophysical logging tools are deployed in a heave-compensated set-up if logging takes place from a heaving vessel. The geophysical logging tools and wireline cable are suspended such that the downhole logging tool can be held stationary with respect to seafloor.

Table A.5 provides example practice for borehole geophysical logging.

The drilling and logging stage can require decisions on feasibility of open-hole logging, multiple sections and multiple runs. A common arrangement is (a) single borehole drilling operation, (b) open-hole borehole, (c) single section and (d) run(s) as required for data types. Moreover, decisions on length of “rat hole” to be drilled should be assessed prior to commencing logging activities.

Table A.5 — Typical tool strings – wireline deployment

Tool string	Typical logging speed m/min	Comments
GR-OT-CAL Natural gamma radiation Orientation and temperature Caliper	3 to 6	Typically, first tool string that is run. Check for borehole conditions
GR-DLL Natural gamma radiation Resistivity	1 to 5	Typically, second tool string.
GR-SON Natural gamma radiation Sonic velocity	1 to 2	Typically, last tool string. SON tool centralized in borehole with bow spring centralizers
SGR Spectral gamma radiation	0,5 to 1	
VSP Vertical seismic	Discontinuous	Separate run of logging tool. Requires acoustic source (e.g. airgun) at or above seafloor
PSSL P and S suspension	Discontinuous	Separate run of logging tool

NOTE 1 Tool string identification implies top to bottom, e.g. GR would be upper sensor for GR-OT-CAL.

NOTE 2 VSP profiling can also be performed by means of the seismic SCPT for measuring the (primary compression) wave velocity v_p (see Nauroy et al. (1998)[147]).

Common termination criteria for borehole geophysical logging are:

— Reaching target of logging coverage or target borehole depth;

— Termination criteria for conventional borehole and casing pipe installation, including adverse borehole conditions (e.g. swelling and caving);

— Inadequate data quality with no improvement expected for any follow-up logging runs;

— Risk of damage to apparatus or safety of personnel, at discretion of equipment operator or as determined by software algorithms;

— Adverse or deteriorating weather conditions.

Logging systems utilizing chemical radioactive sources (e.g. gamma-gamma density and dual neutron) can be considered but are not recommended for marine soil investigation. Particularly, it can be impossible to recover a radioactive tool from the borehole. Any contingency activities for tool loss can far exceed the original cost of borehole geophysical logging.

Pertinent data acquired by a sonic tool, vertical seismic profiler or P and S suspension logger allow estimation of low-strain (dynamic) elastic parameters from P-wave and S-wave velocities.

A.7.6.2 Reporting of results

No additional guidance is offered.

A.8 Guidance on in situ testing

A.8.1 General

No additional guidance is offered.

A.8.2 General requirements for reporting of in situ tests

No additional guidance is offered.

A.8.3 Cone penetration test

A.8.3.1 General

No additional guidance is offered.

A.8.3.2 Equipment

No additional guidance is offered.

A.8.3.3 Test procedures

A.8.3.3.1 Selection of equipment and procedures

No additional guidance is offered

A.8.3.3.2 Calibration and verification requirements

The measuring intervals for calibration of the sensors of the cone penetrometer should be selected to cover the measuring intervals of interest. Considerations for the measuring intervals of interest include expected ground conditions and operational setting.

In most cases, a wide range of expected ground conditions and water depths will dictate the measuring intervals for calibration. For axial force and pressure, this typically implies interval end-points close to the limiting operating conditions or at a selected percentage of survival conditions of the sensors. Common end-points for cone resistance are 50 MPa and 80 MPa. Calibration of subtraction type cone penetrometers typically implies end-points for sleeve friction that are proportional to cone resistance by the ratio of cross-sectional projected area of the cone to the surface area of the friction sleeve (e.g. 1 000 mm^2/15 000 mm^2). For pore pressure, common end-points are 5 MPa, 10 MPa, 20 MPa and 40 MPa, where the higher values would be largely controlled by expected water depths.

For calibration of axial force and pressure, a smaller measuring interval can improve cone penetrometer class and can improve measurement uncertainty for test results within that interval compared to a larger measuring interval for calibration. Most cone penetrometers allow data acquisition to the limiting operating condition, with excellent resolution and with a highly linear response from zero force/ zero pressure to the limiting operating condition. This is regardless of the measuring interval for calibration. This means that the selection of measuring intervals for calibration can consider end-points that are lower than values expected to be measured during cone penetration testing.

A measuring interval of interest of 0^o to 15^o is typically selected for calibration of inclination.

A.8.3.3.3 Cone penetrometer class conformity assessment

No additional guidance is offered.

A.8.3.4 Procedures for testing offshore

A.8.3.4.1 Reference readings - CPTU/CPT in non-drilling mode and in drilling mode – seafloor drilling

The procedure described in this subclause should be applied to cone penetration tests in non-drilling mode and in drilling mode – seafloor drilling.

Documentation of reference readings should be according to one of the following options, see Figure A.2:

a) time stages 1-9 (full cycle);

b) time stages 3-7.

The time stages are as follows:

— Stages 1-2: Initial acquisition of reference readings from all sensors at deck level. The reference readings should be recorded once the output signals from the sensors are stable.

— Stages 2-3: Lowering of cone penetrometer to seafloor.

— Stages 3-4: The cone penetrometer is located at, but not in contact with, seafloor. The reference readings should achieve stable values during this stage.

— Stages 4-5: Penetration into seabed.

— Stages 5-6: Extraction of the cone penetrometer until it is located again at, but not in contact with, seafloor.

— Stages 6-7: The cone penetrometer is located at, but not in contact with, seafloor. For option 2), the reference readings should achieve stable values during this stage.

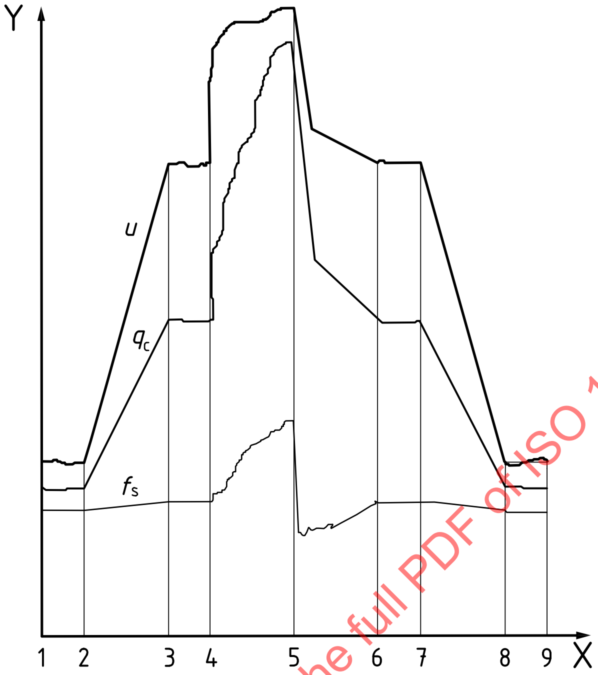
  NOTE Option 1) can be extended to include option 2), so that the better set of reference readings can be selected for compliance to Table 4.

— Stages 7-8: Recovery of the cone penetrometer to deck level. Visual inspection of the cone penetrometer for any damage, soil adhesion to the cone penetrometer, damaged or dirty seals, etc.;

— Stages 8-9: The cone penetrometer is at deck level. The cone penetrometer should remain in a vertical orientation during these measurements. The reference readings should achieve stable values during this stage, after the cone penetrometer is at approximately the same temperature as stages 1-2.

Changes in cone resistance, sleeve friction and pore pressure during stages 2-3 and stages 7-8 can be used to check performance of the cone penetrometer.

For option 1), the difference between stable readings for stages 1-2 and stages 8-9 provides input for conformance to Table 4. For option 2), the corresponding stages are stages 3-4 and stages 6-7.



Key

X	axis shows time	4-5	penetration of cone penetrometer into soil
Y	axis of sensor readings	5-6	pulling out cone penetrometer to seafloor
1-2	reference readings on deck	6-7	reference readings at seafloor
2-3	lowering cone penetrometer to seafloor	7-8	pulling cone penetrometer up to deck
3-4	reference readings at seafloor	8-9	reference readings on deck

Figure A.2 — Scheme for taking reference readings

A.8.3.4.2 Reference readings - CPTU/CPT in drilling mode - vessel drilling

Deck-to-deck recording and reporting of reference readings should be applied to cone penetration tests in drilling mode – vessel drilling. The procedure should include stages 1-2 and stages 8-9 described in A.8.3.3.2. The difference between stable readings for stages 1-2 and stages 8-9 provides input for conformance to Table 4.

Data processing of recorded measurements should consider the following methods:

a) taking sensor offset values at the time of latching of the tool in the bottom-hole assembly;

b) calculating theoretical offset values from fluid pressure derived from combined water depth and depth of cone penetrometer below seafloor.

Methods a) and b) usually result in offset values that are very close to each other. Substantial differences between the two methods should be clarified where necessary.

Technical reasons for differences between results from methods a) and b) include:

— uncertainty margin for the effect of the hydrostatic fluid pressure on the sensor readings [methods a) + b)];

— fluid pressure in the gap of the cone penetrometer not acting on the full theoretical cross-sectional area(s) [methods a) + b)];

— inadequate borehole cleanliness, causing the cone penetrometer to be in contact with the soil at the time of latching of the tool in the bottom-hole assembly [method a)];

— drilling fluid unit weight differing from that of the seawater and the pore fluid [method a)];

— suspended drill cuttings affecting the unit weight of the introduced drilling fluid [method a)];

— significant hydraulic resistance (surging) in the annulus between the drill pipe and the surrounding soil [method a)];

— uncertainty margin for the calculated fluid pressure derived from the combined water depth and depth of cone penetrometer below seafloor [method b)].

A.8.3.4.3 Penetration rate

In some cases, important information can be obtained by carrying out tests at non-standard rates. If such tests are carried out, results should be clearly marked noting that non-standard rates have been adopted.

A.8.3.5 Presentation of results

A.8.3.5.1 Corrected sleeve friction

It is possible to correct the sleeve friction for pore pressure effects if u_2 and pore pressures just above the top of the friction sleeve (u_3) have been measured or can be estimated. Values of f_t can also be estimated from Formula (A.1):

$$f_{\mathrm{t}} = f_{\mathrm{s}} - \frac{(u_2 \cdot A_{\mathrm{sb}} - u_3 \cdot A_{\mathrm{st}})}{A_{\mathrm{s}}} \quad \text{(A.1)}$$

where

f_{t} is the corrected sleeve friction, in MPa;

f_{s} is the sleeve friction according to 8.3, in MPa;

A_{s} is the surface area of the friction sleeve, in mm^2;

A_{sb} is the cross sectional area of the bottom of the friction sleeve, in mm^2;

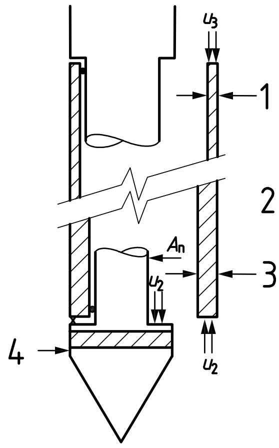
A_{st} is the cross sectional area of the top of the friction sleeve, in mm^2;

u_2 is the pore pressure between the friction sleeve and the cone, in MPa;

u_3 is the pore pressure just above the friction sleeve, in MPa.

NOTE u_3 can be estimated from u_2 using correlations given by Lunne et al. (1997)[141].

Figure A.3 illustrates the correction of sleeve friction due to pore pressure effects.



Key

1 cross sectional area (top) A_{st}

2 surface area of the friction sleeve A_s

3 cross sectional area (bottom) A_{sb}

4 cross sectional area A_c

Figure A.3 — Correction of sleeve friction due to pore pressure effects

A.8.3.5.2 Uncertainty of CPTU/CPT parameters

A.8.3.5.2.1 General

The difference between reference readings can inform about whether a cone penetrometer system is working as expected, but it does not inform about the wider uncertainty of the data gathered. The uncertainty of individual data points of a CPT profile is affected by many factors. These uncertainty factors should be taken into account when deriving geotechnical parameter values from the recorded data. A number of factors affecting cone penetration test uncertainty are discussed in this subclause. Further discussion on uncertainty associated to CPT/CPTU are provided by Soage Santos R.(2022)[157].

To help detecting anomalous data, more than one calibrated cone should be used in similar soil conditions to verify cone resistance.

A.8.3.5.2.2 Soil stratigraphy

For a soil profile of dense sand overlying soft clay, there is more uncertainty in the measurements made in the soft clay than there would be in a soft clay only profile, due to thermal effects and hysteresis effects in the cone penetrometer response. For example, friction in dense sand can typically cause a cone penetrometer to heat by about 1 °C/MPa cone resistance and the resulting heat flux in the cone penetrometer can give an apparent shift in the cone resistance. Ambient temperature compensation systems cannot avoid heat flux effects. Penetration interruption to allow the cone penetrometer to return to stable ambient temperature can mitigate the effect on the cone penetrometer readings. It is also possible to measure temperature in the cone penetrometer and partially correct CPT parameter values for heat flux effects (see 8.3.3.2).

A.8.3.5.2.3 Pore pressure correction

To improve the accuracy of geotechnical parameter values derived from cone resistance measurements, the cone resistance is frequently corrected for the effects on the cone resistance of the porepressure generated by the cone penetration. However, if the pore pressure recorded by the pore pressure sensor is not representative of the water pressure around the cone, the correction applied to the cone resistance will be incorrect. Such error could be due to the filter becoming blocked or the pore pressure sensor can desaturate due to high negative pore pressure generated in the test and therefore not respond representatively as a result of the soil behaviour. The gap behind the cone, over which pore pressure affects the cone resistance, can be unsaturated. Pore pressure in the gap can thus be significantly different from the measured pore pressure. In all these cases, the assumed correction for pore pressure effects on the cone resistance will be erroneous and can cause errors in the derived values of geotechnical parameter.

A.8.3.5.2.4 Sleeve friction

Significant variability is reported in the sleeve friction measurements made by different designs of cone penetrometer that are all in accordance with 8.3.2. This variability is larger than that for cone resistance.

A.8.3.5.2.5 Variations in rate of penetration

It is not uncommon that cone penetration rate vary and are lower (typically) or higher (uncommon) than desired. Cone penetration rate is dependent on applied thrust, which is commonly controlled by the pressure applied in the thrust system.

Cone penetration rate can influence the pore pressure response (drained, partially drained or undrained) of specific soils/weak rocks (e.g. chalks, silt). Pore pressure response will in turn affect cone resistance during penetration. For example, a soil responding undrained during a CPT will typically have lower resistance to cone penetration.

Effects of varying penetration rate on cone resistance in soils with a constant pore pressure response, i.e. either drained or undrained, are limited. DeJong et.al. (2013)[118] provide examples of the effect of variable rates of cone penetration.

A.8.4 Pore pressure dissipation test

No additional guidance is offered.

A.8.5 Ball and T-bar penetration test

No additional guidance is offered.

A.8.6 Seismic cone penetration test

A.8.6.1 General

The project specifications can stipulate SCPT, for example for earthquake studies requiring an approximate average horizontally polarised shear wave velocity for a depth range from seafloor to rock or a competent soil layer.

A.8.6.2 Equipment

Improved quality of derived values of interval shear wave velocity, v_s, can be achieved by using a dual-element seismic cone penetrometer.

A.8.6.3 Procedures for testing offshore

A.8.6.3.1 Source repeatability and ground coupling

Source repeatability can be assessed by:

— comparison of arrival times between stacked and unstacked data, where the offset between relative arrival time should be less than 1 %,

— similarity of amplitude and frequency content between consecutive signals recorded by the receivers.

Activation of the source and signal generation at seafloor level should be verified taking account of factors such as variations in trigger latency and cable length. In those cases, system latency should be measured, and corrections should be applied to the recorded signals. Supplementary timestamps can be considered for the purpose of verification.

Ground coupling between seismic source and seafloor can depend on seafloor conditions. Where applicable, fins or skirts can be used to support coupling between seismic source and seafloor.

A.8.6.3.2 Trigger repeatability

The standard measurement uncertainty for repeatability of trigger latency should conform with Formula (A.2):

$$u_{tr} \leq 0{,}1 \cdot \Delta h_{sr} \tag{A.2}$$

where

u_{tr} is the standard measurement uncertainty for repeatability of trigger latency, in ms,

0,1 is a constant, in ms/m;

Δh_{sr} is the vertical spacing between the seismic receivers, in m, used for obtaining shear wave velocity, v_s.

The method, frequency and results of verification of u_{tr} should be reported as a function of the vertical spacing Δh_{sr}. Trigger latency applies to the trigger system and is defined as the time between the following steps a) to d):

a) initiating a command signal for activating the seismic source, typically by a switch of a data acquisition unit;

b) travel time through one or more cables for transmission of the command signal to the seismic source;

c) activation of the seismic source;

d) actual start of acoustic propagation at the seismic source.

The following test method can be applied for on-site verification of u_{tr}:

1) position the seismic cone penetrometer at a close and fixed distance from the seismic source, where both the seismic source and the seismic cone penetrometer are suspended and submerged in (sea) water;

   NOTE A (sea)water setting promotes a controlled travel path for seismic waves between the seismic source and the seismic cone penetrometer.

2) consecutively activate the seismic source and record the time records for initiating the command signal for activating the seismic source and the subsequent responses of the seismic receivers;

3) estimate trigger latencies from the test records and calculate u_{tr}.

Repeatability of trigger latency is important for single element seismic cone penetrometers. Repeatability of system latency is also important for dual element seismic cone penetrometers, in case of data stacking.

Trigger latency is generally constant for a particular system set-up and environmental setting. A change to any of the system components and settings can change trigger latency. One of the settings is the sampling frequency for records of trigger latency.

A.8.6.4 Presentation of results

Seismic trace analysis can be used to analyse the quality of recorded seismic CPT data and can include:

- signal-to-noise ratio in decibel (dB) units , i.e. 10 x $\log_{10}$ (signal/noise);
- cross-correlation coefficient;
- wave form linearity;
- location-specific considerations affecting data quality.

Methods for seismic trace quality assessment are given in Baziw and Verbeek (2017)[101] and Styler and Weemees (2016)[158].

A.8.7 Other in situ tests

A.8.7.1 General

Other in situ tests include, but are not limited to:

- field vane test;
- flexible dilatometer and/or pressure meter test;
- free fall penetration test;
- electrical conductivity cone penetration test;
- temperature cone penetration test;
- thermal conductivity by probe test;
- hydraulic fracture test;
- pore water/gas sampler with test probe;
- ambient pressure sampler.

A.8.7.1.1 Sources for guidance

Guidance and examples of the application of other available in situ tests are provided in:

- Lunne et al. (1997)[141], Schnaid (2009)[155] and Peuchen and Raap (2007)[153] for overviews;
- Orange et al. (2005)[149] for needle-like CPT, for pore pressure measurement and for performance of dissipation tests;
- Vardon et al. (2019)[166] for thermal conductivity derived from temperature dissipation testing;
- Liu et al. (2022)[139] for thermal conductivity derived from CPT-based heat flow testing;
- Wright and Tan (1991)[169] for hydraulic fracture testing.

A.8.7.1.2 Field vane tests

The FVT should be performed according to ISO 22476-9 or ASTM D2573, where applicable and practicable for deployment in marine soil investigations.

In some cases, shearing rates different from standard rates can be adopted. If such tests are carried out, results should be clearly marked noting that non-standard rates have been adopted. Guidance on effects of varying shearing rates is given in Peuchen and Mayne (2007)[152].

A.8.7.1.3 Free fall penetrometers

The free fall penetrometer described in this subclause is equipped with either a friction cone penetrometer or a piezocone penetrometer at the bottom end. The deployment mode is non-drilling.

NOTE The term "free fall" refers to penetration rates that can be achieved by, for example, (a) a trigger mechanism that determines a drop height above seafloor and that releases the free fall penetrometer and a trailing cable, and (b) lowering the free fall penetrometer by means of a cable on a fast winch.

The free fall penetrometer system should be designed to promote vertical penetration into the seabed. Deviation from verticality should be recorded by means of an inclination sensor positioned in the cone penetrometer.

The following requirements of 8.3 should apply to the free fall penetration test:

a) geometry and the allowable tolerances of the cone penetrometer;

b) cone penetrometer class and calibration of the cone penetrometer;

c) Table 4 for assessment of test category.

The derived values for this test method should include equivalent CPT q_t values normalised to a penetration rate of (20 ± 5) mm/s. The presentation of results should include the derived values at 20 mm depth intervals and a description of the interpretation model for penetration rate effects (see e.g. Young et al., (2011)[172]).

NOTE Vertical penetration rate is commonly derived from 3D accelerometers positioned in the cone penetrometer or close to the cone penetrometer, typically by a combination of data acquisition rates exceeding 1 500 Hz and noise filtering techniques.

A.8.7.1.4 Flexible dilatometer and/or pressuremeter test

Flexible dilatometer and/or pressuremeter tests should be performed according to ISO 22476-4, ISO 22476-5, ISO 22476-6 or ASTM D4719 or ASTM D8359, where applicable and practicable for deployment in marine soil investigations.

Full displacement pressuremeters test should be performed according to, ISO 22476-8, where applicable and practicable for deployment in marine soil investigations.

Whittle (2018)[168] provides an overview of offshore pressuremeter testing.

A.9 Guidance on sampling

A.9.1 Purpose and objectives of sampling

No further guidance is provided.

A.9.2 Sampling systems

No further guidance is provided.

A.9.3 Selection of samplers

A.9.3.1 General

The information in this subclause describes the available tools typically used in a marine soil investigation, and comments on their geometry, applicability and the potential sample quality that can be achieved. It further provides guidance as to what information should be reported on the sampling tool systems.

A.9.3.2 Drilling mode samplers

A.9.3.2.1 Samplers and their applicability to different soils

Table A.6 provides guidance on samplers deployed downhole via drilling mode. This includes the nominal range of penetration into the soil, sampler diameter and applicability in different soil types. The sampler geometries given are industry standard, although other dimensions exist and can sometimes be preferable. The footnotes a) to f) in Table A.6 indicate factors which can contribute to a reduction in the quality of the sample recovered (see Table 9, valid for low to medium OCR marine clays).

Table A.6 — Samplers deployed downhole in drilling mode

Sampler type	Range of penetration m	Sample diameter[a] mm	Applicability for main soil types					
			Very soft to soft clay	Stiff to very stiff clay	Very stiff boulder clay	Loose to medium dense sand/silt	Dense silt/sand (n/a gravel)	Weakly cemented soils and rock
Piston sampler	1 to 2+	44 to 75	A to B	A to B	n/a	A[b]	n/a	n/a
Push sampler, thin-walled	1 to 2	75	A	A to B	n/a	A[b]	B to C[c]	B to C[c]
Push sampler, thick-walled	0,5 to 1	40 to 70	D	D	B[d]	D	C[e]	C[e]
Rotary core sampler	1 to 3	44 to 101[f]	n/a	B to C	A	n/a	n/a	A[b]
Percussion/vibratory sampler	1 to 5	50 to 97	D	B to C	B to C	B to C	A to C	B to C
Hammer sampler	0,1 to 1	40 to 60	D	D	B	C[g] to D	A to C[g]	B to C

NOTE 1 Capital letters denote 'applicability class' where: A = high applicability/recommended, B = medium applicability, C = low applicability, D = very low applicability, and n/a = not applicable.

NOTE 2 Penetration refers to depth below base of borehole.

[a] Industry practice. Larger diameter would normally provide higher sample quality but this should be considered in relation to sample length and sampling technique.

[b] Loss of sample is an issue and can necessitate use of alternative samplers with core catcher.

[c] Only short samples possible, and the leading edge of the cutting shoe is prone to buckling.

[d] Limited penetration and no recovery is likely where gravel and/or cobbles are encountered.

[e] Offers a reduced likelihood that sufficient penetration and recovery will be achieved.

[f] Larger diameters are preferable.

[g] Can be the only way to obtain some recovery.

A.9.3.2.2 Sample tube geometry and dimensions

The following information should be considered for sampling tubes:

a) The thickness of the tube wall should be chosen so that the tube resists distortion when pushed into the soil.

b) The thin-walled tube samplers used (which are commonly used for push and piston sampling, and typically have outer and inner diameters of approximately D_o = 76 mm and D_i = 72 mm, respectively) should meet the following requirements, which apply by analogy to samplers with other internal diameters:

1) The edge taper angle should not exceed 5°;

2) Taper angles between 5° and 15° and area ratios up to 25 % can be used if it is demonstrated that the sample quality is not affected;

3) The area ratio, C_A [where $C_A = (D_o^2 - D_i^2)/ D_o^2$] should be less than 15 %;

4) For tube samplers with C_A exceeding 15 %, the angle of the cutting edge should decrease as the wall thickness increases;

5) Inside clearance $C_i = (D_i - D_c)/D_c$, where D_c is the internal diameter of the cutting shoe, is typically in the range 0 % to 1 % and preferably less than 0,5 %. When assessing the inside clearance, the worst-case manufacturing tolerances should be applied.

A.9.3.3 Non-drilling mode samplers

A.9.3.3.1 General

Table A.7 provides guidance on samplers deployed in non-drilling mode. This includes range of penetration into the soil, sampler diameter and applicability for different soil types, where higher applicability normally means better achievable sample quality. The geometries given are industry standard, although samplers with other dimensions exist and can sometimes be preferable.

Table A.7 — Samplers deployed in non-drilling mode

Sampler type	Range of penetration m	Sample diameter mm	Applicability for main soil types					
			Very soft to soft clay	Stiff to very stiff clay	Very stiff boulder clay	Loose to medium dense sand/silt	Dense sand/silt/gravel	Weakly cemented soils and rock
Piston core sampler with fixed reference to seafloor	10 to 25	100 to 150	A	A to B	n/a	B to C[a]	n/a	n/a
Piston core sampler without fixed reference to seafloor	2 to 50+	70 to 150	B to C	A to B	n/a	B to C[a]	n/a	n/a
Gravity core sampler	2 to 7	75 to 150	A to B	B	D	B to C	n/a	n/a
Vibrocore sampler	3 to 10	75 to 150	C	C	B to C[a]	A to B	A	B to C

NOTE Capital letters denote ‘applicability class’ where: A = high applicability/recommended, B = medium applicability, C = low applicability, D = very low applicability, and n/a = not applicable.

[a] Short samples and limited or incomplete sampler penetrations into the seabed are commonly encountered, particularly in very stiff seabed conditions.

[b] General relevance in all soils, but sample quality is generally poor and volume often varying.

Table A.7 *(continued)*

Sampler type	Range of penetration m	Sample diameter mm	Applicability for main soil types					
			Very soft to soft clay	Stiff to very stiff clay	Very stiff boulder clay	Loose to medium dense sand/silt	Dense sand/ silt/ gravel	Weakly cemented soils and rock
Box core sampler	0,4 to 0,5	200 to 500 X 300 to 500 square/ rectangle	A	C	n/a	C	n/a	n/a
Grab sampler[b]	0,1 to 1	10 to 1 000 square/ rectangle	B	C	D	B	C	D to n/a

NOTE Capital letters denote ‘applicability class’ where: A = high applicability/recommended, B = medium applicability, C = low applicability, D = very low applicability, and n/a = not applicable.

[a] Short samples and limited or incomplete sampler penetrations into the seabed are commonly encountered, particularly in very stiff seabed conditions.

[b] General relevance in all soils, but sample quality is generally poor and volume often varying.

A.9.3.3.2 Piston core sampler with fixed reference to seafloor

Short or long piston core samplers can be designed to give undisturbed samples or high quality intact samples with a fixed reference frame. When pushed at controlled speed or free fall, they differ from gravity and piston core samplers without a fixed reference, since they provide a truly fixed reference for the piston. Penetration and hence recovery ratio can be measured in the best quality samplers, but most frequently the penetration can only be inferred from observation of soil smeared on the outside surface of the sampler.

The following information should be reported for these samplers:

a) precautions taken to reduce inside friction of liners;

b) precautions taken to reduce outside friction of the sampler (cutting shoe and core barrel);

c) reference to piston position during sampling;

d) method of pushing or intruding the sampler into the seabed;

e) method of measuring penetration, including the means for controlling/minimizing any penetration of the seafloor frame into very soft seabed soils;

NOTE Penetration of the seafloor frame will affect data depth uncertainty (see Clause 6).

f) geometry and dimensions of cutting shoe;

g) any additional measurements, such as suction below piston, force to push the sampler into the soil and penetration rate.

A.9.3.3.3 Piston core sampler without fixed reference to seafloor

Short or long piston core samplers can be designed to give reasonable quality intact samples with a relatively stationary piston (e.g. piston fixed to wire from winch on vessel). These samplers operate as free-fall samplers and are triggered when sampler shoe and piston are at the seafloor. They differ from gravity core samplers, since they provide a relatively fixed reference for the piston, although both penetration and recovery are normally not measured. However, as for gravity core samplers, soil smeared on the outside of the sampler can provide an indication of the penetration depth.

The following information should be reported for these samplers:

a) precautions taken to reduce inside friction of liners;

b) precautions taken to reduce outside friction of the sampler (cutting shoe and core barrel), if any;

c) reference of piston position during sampling;

d) geometry and dimensions of cutting shoe.

A.9.3.3.4 Gravity core sampler

The gravity core sampler should provide minimum resistance to the penetration of the unit into the seafloor. The unit should have a streamlined outline and should include stabilizing tail fins. The core cutter head and core catcher should be easily removable, to allow removal of the liner with sample. The barrel should be attached to the tail/weight assembly by a flange that can be opened to assist in retrieval of the core liner when a bent barrel can restrict removal from the base. The top of the barrel should include a non-return valve to reduce sample loss during recovery. For very soft clays, it should be taken into account that non-return valves and core catchers can cause plugging, remoulded soils and poor recovery.

The paid-out wire length should be accurately measured with a wireline counter. The core barrel should normally have a minimum length of 2,0 m. In the case of very hard ground conditions, the equipment should also allow attachment of a 1 m length barrel. The core barrel diameter should be at least 75 mm. The outside diameter of the plastic core liner should equal the inside diameter of the steel core barrel, with a tolerance of 0 mm/-0,5 mm. The winch used for the deployment and recovery of the gravity core sampler should allow continuous downward penetration of the sampler into the soil.

The penetration depth below seafloor is not measured, but soil smearing on the outside of the sampler can provide an indication of the penetration depth.

NOTE A gravity core sampler can plug after 0,5 m to 1 m penetration, depending on soil conditions and inside clearance.

A.9.3.3.5 Vibrocore sampler

Recovery versus penetration should be recorded and reported when using a vibrocore sampler. Prolonged and excessive vibration should be minimized during vibratory sampling, either by using a device that can measure continuously the progress of the sampler into the soil or by limiting and setting the total time of vibration.

The samples recovered with this method are generally disturbed. The tests which can be performed on these samples can be limited to description of the material, classification testing and strength testing on moderately disturbed or reconstituted samples. This method of recovering soil samples is not suitable for some materials, such as calcareous sediments, which are susceptible to cementation or crushing. In case of cementation, early refusal is a concern. In case of grain crushing, the particle shape and size recorded does not necessarily represent the in situ conditions.

A.9.3.3.6 Box core sampler

The box core sampler is designed to recover undisturbed samples or high quality intact samples of soft surficial materials. Box internal dimensions up to 0,5 m × 0,5 m × 0,5 m are commonly used for marine soil investigations, but other dimensions can also be used. The box is mounted on a frame, which is lowered to the seafloor using either the vessel crane or a winch, similar to the launch and recovery of the grab sampler. A surcharge weight is placed on top of the box. A self-releasing trigger mechanism, initiated once the frame reaches the seafloor, allows the box to penetrate into the seabed under its own and surcharge weight. The penetration is limited by a stopper to about 0,5 m depth.

During recovery, a bottom lid is activated and closes the box at the base of the sample to prevent wash-out of the base of the soil sample. Once the box sampler is returned to deck, the bottom lid is replaced by a plate and the box is freed from the frame.

Sub-sampling and testing of the box core sample can typically include:

— sub-sampling;

NOTE 1 Tubes (about 2 to 4) of diameter 50 mm to 250 mm are pushed into the sample. These sub-samples are labelled, sealed and kept for further onshore testing.

NOTE 2 Be aware of compression of sub-samples.

— index testing;

  — sub-sampling for density and water content evaluations,

  — torvane, laboratory vane and pocket penetrometer tests.

— laboratory vane, small scale or miniature penetration testing (e.g. cone, T-bar or ball tests, see B.2.15.8), performed in the recovered box core sample;

— soil identification and description (logging).

Method of tube insertion into the core box and considerations of sample quality and disturbance should be reported. The recovered material should also be identified and described.

The remaining material can be bagged and labelled for onshore laboratory testing.

A.9.3.3.7 Grab sampler

A grab sampler capable of acquiring at least 40 l of sample should be used. The grab sampler should be able to fully seal the sample in order to minimize loss of material fines during recovery to the surface.

The sampled material should be emptied above a funnel over a container capable of holding the quantity of required material from the specified location (e.g. a 200 l drum) while allowing excess water to escape. If steel containers are used, a plastic liner should be used in order to avoid rapid rusting of the drums and hence contamination of the material. Once the sample recovery is completed, the drum should be sealed and placed on palettes for shipment ashore. The palettes should be designed for safe handling and transportation.

The launch and recovery of the grab sampler should be performed using either the vessel crane or a dedicated winch. The dimensions of the grab sampler should suit the purpose of the sampling. . The use of a non-rigid steel wire to deploy the sampler, allowing the operator to 'feel' the seafloor, is preferred to heavy winch wire.

A.9.3.3.8 Information for seafloor samplers

For a non-drilling-mode marine soil investigation, the following information should be reported, where applicable:

a) mass of the equipment in air and submerged weight in water;

b) maximum thrust that can be applied to penetrate into the seabed;

c) equipment handling methods;

d) deck space requirements;

e) summary of the lifting equipment performance characteristics;

f) manufacturer and serial number of the equipment (where available);

g) any operating limitations of the equipment, such as:

— maximum water depth in which it can be operated;

— soil types to which the sampler is best suited;

h) how the sampler penetration is measured/assessed.

A.9.4 Sample recovery considerations

No further guidance is provided.

A.9.5 Handling, transport and storage of samples

A.9.5.1 General

Recommendations in this subclause are primarily for sample tubes of approximately 1 m length. For longer tubes or liners, it is often practical to cut these into shorter lengths, typically 1 m or less.

A.9.5.2 Offshore sample handling

A.9.5.2.1 Extruded samples

Samples in sample tubes with no liner should be extruded in the same direction as they entered the sample tube, to avoid reversal of shear stresses at the sample-tube interface. Prior to extruding samples, the ends of the samples should be subjected to torvane, laboratory vane and/or pocket penetrometer testing, as appropriate.

All samples should be logged and a significant number of colour photographs taken after extrusion. Photography of the samples should be at a single focal length. Photographs should be retained throughout the investigation. Each photograph should include a record of borehole number, depth and sample length. Where applicable, excess drilling fluids should be wiped off and the sample side facing the camera should be gently shaved to display structure. A reference scale and colour chart should also be included in the photograph. Where the sample has been split for examination, additional photographs should be taken of the exposed soil fabric or structure.

After soil identification, description and photographing the sample, the sample should be cut into sub-lengths (sample specimens). Reporting should include a sample record for each specimen with borehole number (or non-drilling sampling number), sequential sample number, sample depth, sample length, soil identification and description. Representative sub-samples should be selected for basic laboratory testing on board the vessel. For each borehole and for each non-drilling sampling activity completed, reporting should include a sample inventory that covers the sample record and onboard laboratory test results.

All specimens not tested on board should be wrapped in plastic cling wrap, followed by aluminium foil. The wrapped sample should then be placed in a cardboard cylinder (or similar) with an internal diameter larger than the sample diameter, and should be sealed with a low-shrinkage wax. The cylinder should be clearly labelled with an indelible marker providing the project identifier, borehole number, sample number, depth and orientation marks. The specimen samples should then be packed in a sturdy container with a lid, lock, and lifting handles or arrangements. Packaging in this container should be such that samples are protected against shock and restrained against movement. The dimensions and final mass of the container should be within a range that can be safely handled. The container should be clearly labelled with indelible marker providing a project description, borehole number (or non-drilling sampling number), depths and number of samples contained therein. The top of the container should be clearly marked as such. These markings should be made on the top of each box and on at least one side. A sample list should be placed inside each container.

Samples subjected to on-board testing should be retained and bagged after testing. These samples should be clearly labelled, using an indelible marker, with borehole number (or non-drilling sampling number), sample number and depth.

A.9.5.2.2 Unextruded samples

If a non-cohesive or very soft cohesive sample is judged to be better left in the sample tube (liner) and suitable for extrusion under controlled conditions onshore, then such samples should be end-logged. Cohesive samples not extruded should be subjected to torvane, laboratory vane and/or pocket penetrometer testing, as appropriate. If appropriate, sub-samples can be removed from the ends and subjected to on-board testing (water content and density measurements).

Where the sample is flush with the end of the sample tube (liner), an end cap should be placed on the sample tube (liner) and should be sealed against moisture loss using insulation tape. Any gap between sample and sample tube (liner) end should be filled with microcrystalline low-shrinkage wax or other suitable spacer material before placement of the end cap. Mechanical O-ring sealing devices can also be substituted for sealing in place of microcrystalline wax, as long as a seal can be obtained between the O-ring and the inner diameter of the sample tube (liner). Both ends of the sample should be prepared and cleaned prior to tightening the mechanical O-ring device.

The sample tube (liner) should be clearly marked with indelible ink to include project identifier, borehole number (non-drilling sampling number), sample number, depth to the top of sample, date of sampling and orientation. The sample tubes (liners) should be stored vertically in a suitable container. The dimensions and final mass of the container should be within a range that can be safely handled. Each sample should be laterally restrained in the container against movement and padded for shock. The containers should be marked with indelible ink providing the project name, box number and depth ranges included. The top of the container should be clearly marked as such. These markings should be made on the top of each box and on at least one side. A sample list should be placed inside each container.

Unconventional soils are often left unextruded in the sample tubes (liners) for delivery to the onshore laboratory. For example, a common practice in Australia, where calcareous soils predominate, is to minimize the amount of material that is extruded offshore. In other regions, where more sandy and strongly cemented calcareous soils predominate (e.g. the southern Mediterranean Sea and the Arabo-Persian Gulf), the practice is to extrude the samples offshore.

A.9.5.2.3 Rotary core samples

Handling of rotary core samples recovered in full-round liner material can be performed according to A.9.5.2.1. This subclause covers handling of extruded samples.

Core samples can be cut with either a double-tube or a triple-tube arrangement. If triple tubes are used, then the inner barrel of the inner tube is usually comprised of steel splits or other suitable full-round liner material. Once the core sample is on deck, the inner tube should be carefully removed and placed into a plastic split tube which has the same internal diameter as the core diameter (if steel splits are used), or placed directly onto a table equipped with cradles of similar external diameter as the liner. The core sample should then be identified and described (logged).

After identification and description of the core sample, the top half of the plastic split tube should be carefully placed on the core and the two halves should be secured using electrical insulation tape. The plastic split liner should then be clearly marked with indelible ink providing the project identifier, core sample number, top and bottom of the core sample, depths, date of coring and orientation. If a full-round plastic liner was used in capturing the sample, it can be transferred directly into the core sample box. The core sample should then be placed in two layers of heavy-duty lay-flat polyethylene tubing, cut in lengths long enough to allow the ends to be wrapped over and sealed with electrical insulation tape at both ends to minimize moisture change.

The core samples should be packed into specially designed boxes, with spacers between each core sample run, on which core sample run number and depth should be marked with indelible ink. The spacers should also be used to indicate depths over which sample loss occurred. The core sample boxes should have lids that can be securely attached and should be of dimensions and mass that can be safely handled. The core sample boxes should also be marked with indelible ink providing the project name, borehole number, core runs and depths included. These markings should be made on the top of each box and on at least one side.

A.9.5.3 Offshore storage

The sample containers on board the vessel should be stored away from direct sunlight and exposure to the elements. Storage in rooms adjacent to heavy engines, generators or other installations that generate excessive vibrations, should be avoided. Intact samples should be stored vertically with the same orientation as the soil had in the ground. If samples are very sensitive to vibration (e.g. weakly cemented sands), damage or deterioration of the samples from vibration should be avoided, also by considering precautions such as vibration isolation and special padding at the bottom of the sample boxes.

For seafloor drilling, exposure of samples to excessive vibrations during temporary sample storage on the seafloor drill rig should be avoided.

Special storage should be considered for materials intended for environmental, geohazard studies or other work.

A.9.5.4 Onshore transport, handling and storage

If the sample boxes are stored temporarily onshore prior to transportation to the testing laboratory, then such storage should be under cover (indoors), frost-free and secure.

The containers with intact soil samples should be transported to the onshore laboratory with caution and should be handled with care. Precautions should be made to prevent shock and impact loading to the soil samples during handling of the boxes. The sample containers should be stored with the samples vertically where the samples have the same orientation as the soil had in the ground. However, horizontal storage of the samples can also be adequate, especially for firm and stiff soils, rock or materials intended for classification testing and tests on reconstituted samples.

All samples should be transported cushioned on an airbed truck by road to minimize vibration damage to the samples. Air freight should be used for long-distance transportation of the samples, in which it should be avoided that the samples are subjected to temperatures lower than 3 °C and higher than 25 °C.

A.10 Guidance on laboratory testing

A.10.1 General

Reference is made to Annex B for guidance on laboratory testing.

A.10.2 Project specifications

No additional guidance provided.

A.10.3 Presentation of laboratory test results

No additional guidance provided.

A.10.4 Instrumentation, calibration and data acquisition

No additional guidance provided.

A.10.5 Preparation of soil specimens for testing

No additional guidance provided.

A.10.6 Evaluation of intact sample quality

No additional guidance provided.

A.11 Reporting

A.11.1 Definition of reporting requirements

The reporting should be organized in chronological order, covering:

a) field operations and preliminary results;

b) derived values of geotechnical parameters and final results;

c) data interpretation and evaluation of derived values of geotechnical parameters.

An example reporting format is presented in Table A.8. This example format suits portable digital format (.pdf) and/or paper print.

The contents of each report should be presented in a logical and clear manner.

The actual scope of reporting, which can be limited to the field operations or can include data interpretation and evaluation of derived values of geotechnical parameters, can be presented in a stand-alone report or in separate reports (in line with the project specifications).

Depending on the project-specific requirements and the type of marine soil investigation, the report can provide support to more or less complex geotechnical issues. Examples of these geotechnical issues include:

— comprehensive marine soil investigation for detailed design of the foundations for a gravity-based structure, piled jacket platform or similar offshore structure of relatively large dimensions;

— marine soil investigation of regional character in an early project phase (e.g. for a new development site for which the type and location or number of platforms are not fixed at the time of the investigation or for a multi-facility site like an offshore wind farm);

— marine soil investigation with limited scope for surveying a site already covered by a previous investigation, or for a small facility such as a small subsea structure with shallow foundations;

— marine soil investigation along a pipeline or subsea power cable route, where a limited penetration and long distances are specific to the coverage requirements;

— marine soil investigation for geohazards identification purposes.

Any active geological processes or geohazards can affect the geotechnical parameter values or the integrity of the foundations for the planned structure. Such geohazards can consist of seabed crust, past seafloor instability, presence of gas in the pore water or shallow soils (with special attention to any effect of pore pressures in excess of hydrostatic, and the effect of gas on the soil behaviour), shallow water-flow sands, mass transport deposits, hydrates, permafrost, etc. The extent of known geohazards should be clearly specified and defined in the scope of work, since this can require different information to be obtained than obtained from a conventional marine soil investigation.

A.11.2 Presentation of field operations and factual data

A.11.2.1 Field operations and preliminary results

Reporting of the field operations and preliminary results (see NOTE) should comprise a description of the drilling, in situ testing and sampling techniques used. This reporting can consist of:

— description of the marine soil investigation platform (e.g. vessel/rig);

- drilling equipment and procedures, with particular emphasis on the following topics:
  - log of drilling performance, to the extent needed for evaluating the soil conditions;
  - any change in mud composition or density (as appropriate);
  - possible influence of the drilling conditions on the quality of in situ testing and sampling results in the various soils encountered;
- verification checks documentation and/or calibration reports of in situ testing and laboratory testing equipment;
- information on positioning operations, equipment and procedures, with proper documentation of verification checks of positioning equipment;
- log of daily field activities;
- preliminary results (see NOTE) of in situ tests and offshore laboratory tests;
- descriptions of borehole geophysical logging operations and results.

NOTE Where the scope of a marine soil investigation is limited to field work only, the results reported together with the field operations will be the final results.

A.11.2.2 Derived values of geotechnical parameters and final results

After verification of field data and completion of onshore laboratory tests, factual reporting of the geotechnical parameters and final results will usually supersede the previous reporting of preliminary field results.

In addition to documenting each individual test, geotechnical logs and soil profiles should be reported by compiling the main soil data and results in an overview format. Such geotechnical logs and soil profiles should typically comprise the following information, including parameter values plotted versus depth:

a) description of the soil in each geotechnical formation;

b) water content and plasticity parameters;

c) soil unit weight;

d) strength properties within the soil strata, consisting of:

1) undrained shear strength of clays, by correlation with in situ tests, and/or for various types of laboratory tests, with both intact and remoulded strengths;

2) relative density of sands, as derived from the in situ CPTU cone resistance, for example.

The following soil properties and geotechnical parameters are typically presented and plotted versus depth:

- soil description;
- appropriate soil classification data, i.e.:
  - water content;
  - plastic and liquid limits, plasticity index and liquidity index;
  - particle size distribution;
  - soil unit weight;
  - density of solid particles;

  - carbonate, organic and salt contents (where relevant);
  - in sand layers, maximum and minimum void ratios, in situ density and relative density derived from laboratory tests or in situ tests;
  - in clay layers, index undrained shear strength from pocket penetrometer, fall cone, torvane or laboratory/miniature vane tests (as applicable);
  - in situ temperatures (where applicable);
- effective overburden stress, p_0' or σ_{v0}';
- in situ pore water pressure, u_0, including any excess pore pressure (if relevant);
- preconsolidation stress, σ'_p, over-consolidation ratio, OCR and yield stress ratio, YSR;
- small strain shear modulus, G_{max};
- coefficient of consolidation, c_v, and compression index, C_c (if relevant);
- for sand layers, effective friction angle, ϕ', typically using a laboratory test method as reference (e.g. triaxial compression/extension tests, plane strain tests or shear box tests, as applicable);
- for clay layers, undrained shear strength typically using a laboratory test method as reference (e.g. unconsolidated UU triaxial tests, s_{uUU}; isotropically or anisotropically consolidated CIU or CAU triaxial compression/extension tests, s_{uC} and/or s_{uE}; DSS tests, s_{uD}; with estimate of strength anisotropy, and remoulded shear strength with sensitivity, S_t).

NOTE Soil properties and parameter values can be derived from borehole geophysical logging, in situ test results and/or laboratory test methods. A reference laboratory method is typically considered for the purpose of parameter definition.

Anomalous results for a given stratum when compared with results from multiple methods capable of deriving the same parameter should be anticipated. Such anomalous results should be identified. An evaluation should be performed for assessing whether these results are representative of the soil conditions or not.

A.11.3 Data interpretation and soil parameters

Interpretation of marine soil investigation data and results should be reported, including methods and procedures used for evaluation of the soil parameter values.

The following aspects should be considered for geotechnical logs in cases of stratigraphic schematization and subdivision into geotechnical formations:

a) strata in which parameter values vary only slightly and occasionally can be considered as one formation;

b) a sequence of thinly bedded strata with differing geotechnical composition and/or mechanical properties can be considered as one formation only if the overall behaviour is relevant for the design case under consideration and if the behaviour can be adequately represented by the geotechnical parameters selected for the formation;

c) when deriving the boundary between different formations, linear interpolation can be applied between the marine soil investigation points, provided the spacing is sufficiently small and the local geological conditions are sufficiently uniform (should be justified and documented).

Stratigraphic schematization interpreted for the geotechnical logs can be compared for consistency with available marine geophysical investigation data. The observations or measurements during drilling can also be used as an indicator in determining boundaries between different strata.

Depending on the planned type of structure and the foundation design loading case under consideration, different requirements can apply for the evaluation of the geotechnical parameters. These requirements can include:

— detailed characterization of the top 1 m of soil, of particular concern for pipelines and risers;

— evaluation of soil properties with depth for the design of driven piles, with particular emphasis near the pile toe level for end-bearing capacity in a layered soil profile;

— evaluation of cyclic soil properties and permeability of strata for facilities under various periods and rates of loading, such as undrained bearing capacity and shear stresses in the soil caused by cyclic loading of a gravity-based structure, mobilization of passive suction for skirted foundations or suction piles and effect of earthquake loading;

— in unconventional soils (see Table A.3), care should be taken when interpreting the data derived from test methods outside of the intended scope of the particular test method.

Table A.8 — Example reporting format and contents (see NOTE)

Field operations and preliminary[a] results		**Derived values of geotechnical parameters and final results**		**Data interpretation and geotechnical parameters**
	→		→	
Executive summary		Executive summary		Executive summary
Investigated points, with maps and coordinates		Investigated points, with maps and coordinates		Investigated points, with maps and coordinates
List of symbols and terms used		List of symbols and terms used		List of symbols and terms used
Scope of field operations, with description of the marine soil investigation platform (e.g. vessel/rig)		Scope of field operations, with description of the marine soil investigation platform (e.g. vessel/rig)		Summary of soil conditions and stratigraphic schematization
Log of drilling operations (when used for soil data evaluation)		Final geotechnical logs and soil profiles, with proposed stratigraphic schematization		Intended structure(s) and design case(s) under consideration
In situ testing operations, procedures and preliminary[a] results		In situ testing procedures and final results		Evaluation of data and results, with interpretation methods and procedures
Sampling operations, procedures and preliminary[a] results		Sampling procedures and final results		Presentation of the ground model and integrated geoscience study results
Field laboratory test results		Laboratory testing procedures and final results		Recommended representative geotechnical parameters
Preliminary[a] geotechnical logs		Other test results (e.g. MSCL or X-ray logging, chemical tests, geological tests)		Geological features, faults and other geohazards of geotechnical concern
Inventory of recovered samples and proposed onshore laboratory test programme		Other data and results (e.g. geophysical data, results of previous marine soil investigations)		Recommendations on additional soil data possibly needed

[a] Where the scope of a marine soil investigation is limited to field work only, the results reported together with the field operations will be the final results.

NOTE Alternative reporting formats can also be used, including contractor's normal practice. For each project the final reporting structure can be adjusted by deleting inapplicable sections or by adding new sections.

Table A.8 *(continued)*

Log of daily field activities List of parties involved in soil investigation Water depth and tidal measurements Documentation of verification checks, equipment calibration, and metrological confirmation References	References	References
[a] Where the scope of a marine soil investigation is limited to field work only, the results reported together with the field operations will be the final results. NOTE Alternative reporting formats can also be used, including contractor's normal practice. For each project the final reporting structure can be adjusted by deleting inapplicable sections or by adding new sections.		

A number of geotechnical parameters are independent of the type of structure for specific loading cases. On the other hand, specific definitions of a soil parameter (e.g. undrained shear strength of clay, effective friction angle of sand) can be required for specific loading cases or geotechnical issues to be addressed (such as bearing capacity, calculation of settlement, displacement or prediction of installation behaviour). Therefore, the list of parameters that should be reported will vary from case to case and should be defined in the project specifications.

Averaging of mechanical properties over a sequence of strata for foundations mobilizing a large volume of soil can be useful, but should be used with caution since averaging can mask the presence of a weaker zone. Situations where it is important that weak zones are identified can include:

— horizontal sliding modes for a shallow foundation;

— assessment of punch-through risk for a jack-up spudcan;

— axial bearing capacity of piles where end bearing has a significant contribution;

— areas where slope stability is a concern.

Recommendations regarding any necessary further marine soil investigation can be included in the report, with comments justifying the need for this additional work.

Annex B
(informative)

Laboratory testing

B.1 General

This annex covers conduct of laboratory tests as part of marine soil investigations and supplements the requirements in Clause 10. This annex focuses on testing of saturated soil samples. Some reference standards for rock sample testing are suggested in B.12.

B.2 Classification and index tests

B.2.1 Soil identification and description

Soil description can be performed in accordance with ISO 14688-1 or ASTM D2488.

The soil description should include information on, where appropriate:

a) main soil types with principal and secondary fractions (with appropriate mention of estimated carbonate and organic content, where significant);

b) undrained shear strength (clay) and particle size (sand);

c) degree of cementation or weathering;

d) structure, texture or other relevant description;

e) colour, preferably with reference to a soil colour chart (e.g. Munsell chart);

f) shape, angularity and mineral composition of coarse or very coarse particles;

g) miscellaneous, including special features, e.g. presence of siliceous-calcareous ooze and unusual odours.

B.2.2 Soil classification

Soil classification can be performed in accordance with ISO 14688-2 or ASTM D2487.

NOTE 1 Different classification systems can provide different results. For example, when comparing GOST and ASTM systems, there are key differences in definitions of soil type according to grain size and plasticity. In addition, the GOST system is to some extent based on the genesis of the material, while this is not an element of the ASTM system. See GOST 25100.

NOTE 2 Unconventional soils can require the use of a more specific classification system, for example carbonate material can be classified according to Clarke and Walker (1977)[114] and frozen soils can be classified according to ASTM D4083.

B.2.3 Sample photograph

Sample photographs should be in focus and of high quality. Additional requirements for taking sample photographs and, if relevant, photographs of test specimens, should be given in the project specifications.

B.2.4 Non-destructive sampling logging

Radiography can be used to visually evaluate the quality of soil samples, the layering of the soil and the presence/quantity of gravel, cobbles and other inclusions in the sample, as described in ASTM D4452.

The presentation of results from radiography should include:

a) location and dimensions of samples radiographed;

b) scale and top/bottom markings;

c) description of X-ray set-up;

d) sample radiographs.

It is also possible to obtain CT scans of soil samples that are contained in non-metallic sampling tubes or liners. The use of CT scanning technology allows multiple visual images to be obtained along the length of a sample, as with radiography, but in addition can provide non-intrusive cross-sectional images. The procedures used for conducting CT scans of soil samples and the presentation of CT scan results should be documented in the same manner as those for radiography of soil samples, as listed above, and should be reported (when required).

Non-destructive core logging techniques can provide semi-continuous logging of various soil properties, such as p-wave velocity, electrical resistivity and magnetic susceptibility.

B.2.5 Water content

The water (moisture) content, w, should be determined in accordance with ISO 17892-1 or ASTM D2216.

B.2.6 Liquid and plastic limits

The liquid and plastic limits (Atterberg limits), w_L, and, w_P, should be determined in accordance with ISO 17892-12 or ASTM D4318.

NOTE ISO 17892-12 allows use of either the 80 g/30° fall cone (20 mm penetration) or the 60 g/60° fall cone (10 mm penetration), as both methods tend to result in the same w_L value. Additionally, there are other standards that use fall cones that differ in dimensions, mass and/or cone angle from the two fall cones specified by ISO 17892-12 and they give different results.

The method described in ISO 17892-12 uses the fall cone device for determination of the liquid limit, while method described in ASTM D4318 uses the Casagrande cup. The two methods can produce different results. For w_L values less than approximately 100 %, the ISO fall cone gives higher values in comparison with the ASTM Casagrande cup (up to about 4 % to 5 % water content at low w_L values). For w_L values greater than approximately 100 %, ISO fall cone values are less than for the ASTM Casagrande cup values, and become significantly less for high to very high w_L values.

For determination of the plastic limit, both ISO 17892-12 and ASTM D4318 use the same procedure.

The description of the test procedure should state whether the material was dried prior to the test, and if so, by what method. The sample should not be dried before testing. It should be reported whether coarse material has been taken out prior to testing. In this case, the sieve size used to separate the material should be documented. If water has to be added, distilled water should be used even if the in situ pore water contains salt.

B.2.7 Bulk density of soil or soil unit weight

The bulk (total or wet) density of soil, ρ, reported in kg/m^3, should be determined in accordance with ISO 17892-2 or ASTM D7263.

The unit weight for fully saturated samples can be estimated using the measured water content and the derived or assumed particle density of soil grains and salinity of the pore fluid.

The term ‘unit weight’, γ, is commonly used in practice. Soil unit weight is given in kN/m^3. The acceleration due to gravity, g (= 9,81 m/s^2), is used to convert from units of mass to units of force (weight).

B.2.8 Particle density of soil

The particle density of soil grains, ρ_s, should be determined in accordance with ISO 17892-3 or ASTM D854.

The description of the test procedure should state whether the material was dried prior to the test, and if so, by what method. The sample should not be dried before testing. As an alternative to reporting particle density, the unit weight of solid particles, γ_s, can be reported in kN/m^3, or the specific gravity of solid particles, G_s, can be reported ($G_s = \gamma_s/\gamma_w$, where γ_w is the unit weight of distilled water at +4 °C, 9,81 kN/m^3).

In rare cases where the soil reacts with water or when the use of water is not appropriate, a gas method such as that outlined in ASTM D5550 should be used.

B.2.9 Maximum and minimum index densities

The maximum and minimum index dry densities (ρ_{max} and ρ_{min}) should be determined in accordance to the methods developed with consideration to offshore use as described in Knudsen et al. (2020)[133]. If sufficient sample mass is available to meet ASTM test requirements, ASTM D4253 can be considered for ρ_{max} and ASTM D4254 can be considered for ρ_{min} instead.

The maximum and minimum index dry densities (ρ_{max} and ρ_{min}) of a material are the densest and loosest states, respectively, that can be produced using a standard laboratory procedure that minimizes particle segregation and crushing. The test specimen particle size distribution should be determined before and after conduct of the maximum index density test to check for possible particle crushing. If the amount of grain crushing and/or increase of fines content is greater than 5 %, it is possible that the maximum index test result might not be valid. The maximum and minimum index dry densities are usually determined for coarse soils. It is also common to present the results in terms of minimum and maximum index void ratios (e_{min} and e_{max}).

NOTE 1 For some types of sands, it is possible to prepare a laboratory test specimen (without grain crushing) to a higher density than that determined according to the recommended test method for the maximum index density. In such cases the computed relative density of the test specimen will be greater than 100 %.

There are other methods for determining the maximum and minimum index dry densities, and such methods can give different results for the same material. The selected testing equipment and procedure should be reported as well as results obtained using the selected equipment and procedure on a reference sand.

NOTE 2 A reference sand is maintained by a laboratory typically for quality control purposes.

B.2.10 Particle size distribution

B.2.10.1 Sieving and sedimentation

The particle size distribution of soils should be determined in accordance with ISO 17892-4 or ASTM D6913 (for sieve analysis) or ASTM D7928 (for hydrometer analysis).

The following information should be reported:

a) use of dry or wet sieving for coarse or very coarse- soils;

b) sedimentation method used for fine soils;

c) whether deflocculation agents were used;

d) whether the material was dried prior to testing and, if so, by what method.

NOTE 1 Some specific soils that are difficult to deflocculate, such as highly plastic deepwater clays of West Africa, can require the use of chemical dispersive agents [see Thomas et al. (2007)[162]].

NOTE 2 The fines content is defined in some practices using the 0,063 mm sieve (e.g. ISO 17892-4) whereas others define it using the 0,075 mm sieve (e.g. ASTM D6913). In some projects, especially international ones, using both sieves in performing the particle-size distribution test can be considered.

The particle size distribution should be reported on a semi-logarithmic plot showing particle size (log) versus percentage (by mass) finer than the particle size.

B.2.10.2 Particle image analysis

The particle size distribution of soil as determined by image analysis methods should be determined in accordance with ISO 13322-1 or ISO 13322-2. The results of these methods should be calibrated against the methods of ISO 17892-4 or ASTM D6913 or ASTM D7928.

Calibration of methods should consider the limitations and inaccuracies of analysis by sieving and/or sedimentation analysis.

NOTE Image analysis can provide data that are additional to particle size distribution, e.g. particle shape.

B.2.11 Angularity

Angularity of sand and gravel particles should be determined by the method described in ISO 14688-1, Lees (1964)[138] or Pettijohn (1957)[151].

ASTM D2488 gives guidance for performing a visual description of the angularity of soil particles.

B.2.12 Organic content

The organic content of a sample should be determined in accordance with ISO 10694 or ASTM D2974.

The test methods described in ISO 10694 and ASTM D2974 estimate the organic content by determining the loss of mass on ignition of a test specimen at a controlled temperature. Other suitable test methods can be used. For example, organic content can be determined from the mass loss on treatment with hydrogen peroxide (H_2O_2). This method provides a more specific determination of the organic content. Another method of classifying clays and silts as organic is to compare results of the liquid limit test performed on a sample after oven drying with that determined before oven drying as described in ASTM D2487.

NOTE For some specific soils, such as Gulf of Guinea clays which contain a high fraction of kaolinite that decomposes into meta-kaolin when heated and loses mass in the form of water, the loss-on-ignition method can provide an over-estimated organic content. In such cases, deriving the organic content from the total organic content (TOC) is recommended [see Thomas et al. (2007)[162]].

B.2.13 Carbonate content

The carbonate content should be determined in accordance with ISO 10693 or ASTM D4373.

NOTE 1 ISO 10693 uses a gas volume method, while ASTM D4373 uses a gas pressure method.

NOTE 2 Dreimanis (1962)[121] describes a volumetric method that uses the Chittick apparatus and allows for determination of the total carbonate content and the individual amounts of calcite ($CaCO_3$) and dolomite ($MgCaCO_3$).

The carbonate content is expressed as a percentage of the oven-dry mass of the test specimen.

The methods do not distinguish between different carbonate species. Results are reported as the percentage calcite equivalent. Detailed distinction among different carbonate species requires quantitative chemical analysis testing and/or petrographic analysis.

B.2.14 Soluble salt content

Properties determined for classification and indexing (e.g. water content, void ratio) can need correction for pore fluid salinity.

NOTE Noorany (1984)[148] and Kay et al. (2005)[132] provide examples of correction.

The soluble salt content of a sample's pore fluid should be determined according to ASTM D4542 using a refractometer method. ASTM D4542 also describes equipment and procedures for extracting a pore fluid sample from a soil sample. The time period between sampling and testing should be minimized, due to potential chemical changes that can occur within the soil sample.

Resistivity (conductivity) methods can also be used to estimate the pore fluid salinity, providing that a pore fluid specimen large enough for testing can be collected.

B.2.15 Undrained shear strength index tests

B.2.15.1 General

An estimate of undrained shear strength (s_u or c_u) can be determined for cohesive soil samples using a variety of methods. The sample orientation (vertical or horizontal) should be specified for the test. All tests should be accompanied by a determination of the water content of the test specimen or the soil immediately adjacent to it.

While strength index test devices described in this subclause are popular in practice, these tests use fast shear rates with different modes of shear, and the results are greatly affected by sample disturbance. Undrained shear strength profiles developed using these devices often show significant scatter.

B.2.15.2 Fall cone test

The fall cone test should be performed in accordance with ISO 17892-6.

NOTE If a different standard is used to conduct the fall cone test, documentation of the equipment, test procedure and calibration used is especially important, as there are differences in the cone calibration factors between some standards.

The mass and tip angle of the cone used should be reported. The cone surface should be smooth and clean. Routine checks of the cone apex should be made at frequent intervals. The test should preferably be performed while the specimen is still inside the sampler (or within a slip ring) to provide lateral confinement, and to minimize the effect of elastic vertical compression caused by the impact of the cone.

At least three readings should be performed on each specimen. The average of these readings should be taken as the actual reading. Tests should not be conducted close to the side wall of the rigid container. It should be ensured that the zone of influence from one test does not interfere with the others.

B.2.15.3 Pocket penetrometer test

In the pocket penetrometer test, the undrained shear strength is estimated from the force required to penetrate a steel cylindrical plunger (or an adaptor) a fixed distance into a flat soil surface, using a calibrated compression spring.

Adaptors with different diameters mounted on the steel rod can extend the use of the penetrometer to a wide variety of shear strengths. Each adaptor has a specified calibration factor. The adaptor used should be reported. The fixed depth of penetration of the rod or of the adaptor should equal the rod diameter.

The pocket penetrometer should be pushed into the test specimen with a push-in time of about 1 s. The undrained shear strength is estimated as the indicated stress divided by 2. The test should preferably be performed while the specimen is still inside the sampler (or within a slip ring) to provide lateral confinement.

At least three readings should be performed on each specimen. The average of these readings should be taken as the actual reading. Tests should not be conducted close to the side wall of the rigid container. It should be ensured that the zone of influence from one penetration does not interfere with the others.

NOTE Each pocket penetrometer adaptor has a specified conversion factor. The conversion factors can differ among manufacturers.

B.2.15.4 Torvane test

The torvane test can be used to determine the undrained shear strength on a flat surface of a cohesive soil sample in accordance with ASTM D8121/D8121M.

The area of the sample should be at least twice the area circumscribed by the torvane shear blades. The test should preferably be performed while the specimen is still in the sampler (or within a slip ring) to provide lateral confinement. Tests should not be conducted close to the side wall of the rigid container. It should be ensured that the zone of influence from one test does not interfere with the others.

The device should indicate undrained shear strength directly from the rotation of the torsion spring. Adaptors of different dimensions can be mounted on the original vane to accommodate a wider range of shear strengths. The adaptor used should be reported.

B.2.15.5 Miniature vane or laboratory vane

The miniature or laboratory vane (motor- or hand-operated) test should be performed in accordance with ASTM D4648.

NOTE 1 BS 1377-7 is similar to ASTM D4648 with the exception of a significant difference in the recommended rate of vane rotation. For ASTM D4648 it is 60°/min to 90°/min, while for BS 1377-7 it is 6°/min to 12°/min. It is thus important to report the nominal rate of rotation used.

The vane blades should be pushed into the centre of the soil while confined in the sampler (or within a liner) to a minimum depth equal to twice the height of the vane blade, so that the top of the vane blade is embedded at least one vane height.

If the miniature vane is also used to determine the remoulded undrained shear strength (see B.2.15.9), the number of rotations of the vane used for remoulding the specimen should be reported.

Several different ways of determining the undrained shear strength are possible using the miniature vane, consisting of:

a) intact: undrained shear strength as determined on an intact specimen;

b) intact – residual: residual shear strength determined post-peak during initial shearing of an intact specimen;

c) intact – vane-remoulded: determined after a minimum of five to 10 rapid rotations of the vane after completion of the intact test;

d) hand-remoulded: steady-state (post-peak, if it exists) resistance of a hand-remoulded test specimen;

e) hand-remoulded – vane-remoulded: steady-state resistance of a hand-remoulded specimen determined after applying a minimum of 5 to 10 rapid rotations of the vane.

NOTE 2 If the intact strength was determined on a material that suffered sample disturbance, then it is likely to be too low.

NOTE 3 Differing values of the remoulded shear strength are often obtained from the different methods listed above [c) versus d) versus e)].

The vane blade dimensions and specific laboratory vane test(s), as listed in a) through e) above, performed should be reported in the presentation of results.

B.2.15.6 Unconfined compression test

The UCT should be performed in accordance with ISO 17892-7 or ASTM D2166.

It should be reported in the presentation of results whether the test specimen was trimmed to a diameter smaller than the sample diameter (see 10.4) and whether any potential dilation was observed in relation to the trimming. The initial ratio of specimen height to diameter should equal two. The rate of strain should be in the range of 0,5 % to 2,0 % per minute. The rate of strain used should be reported.

The results should be reported in the form of a curve plotting shear stress versus axial strain. The test results should include a sketch or description of the failure shape of the specimen.

The specimen should be split open. A soil description (see B.2.1) should be performed. Any observed intrusions and/or structure should be highlighted (e.g. slickensided, platey, blocky).

B.2.15.7 Unconsolidated undrained triaxial compression test

The UU triaxial compression test should be performed in accordance with ISO 17892-8 or ASTM D2850.

Normally, no pore pressure measurements are required, and thus porous discs and filter papers are not required.

After application of the confining stress, the specimen should be allowed to stabilize under undrained conditions for approximately 10 min before static shearing starts. The rate of axial strain should be in the range of 0,5 % to 2,0 % per minute. The confining stress used and the rate of shear should be reported.

The results should be reported in the form of a curve plotting either axial stress or shear stress versus axial strain. The test results should include a sketch or description of the failure shape of the specimen.

The specimen should be split open, a soil description (see B.2.1) should be performed and the water content determined. Any observed intrusions and/or structure should be highlighted (e.g. slickensided, platey, blocky).

B.2.15.8 Miniature penetration tests

B.2.15.8.1 General

Miniature penetration tests covered by this subclause are the MCPT, the MTPT and the MBPT. The test methods are generally similar to the corresponding in situ test methods given in Clause 8, except that the tests are conducted in a soil sample at atmospheric conditions.

B.2.15.8.2 Specimen preparation

Undisturbed or intact soil samples are commonly selected as specimens for miniature penetration tests. Tests on compacted, reconstituted and remoulded specimens can also be performed. A soil specimen typically has a height between 300 mm and 600 mm and a width or diameter of > 300 mm. The specimen should be confined by a sampler (e.g. box core sampler) or by a sample liner, except for the upper surface from which the penetration starts.

B.2.15.8.3 Equipment

The recommendations for the apparatus are as follows:

The thrust machine should provide a constant rate of penetration into the soil at a rate nominally of (20 ± 5) mm/s. The additional ability to perform tests under load-control can provide valuable information. Minimum limit values for axial force or penetration resistance that can be overcome can be given in the project specifications. Common apparatus for miniature penetration testing allows overcoming a penetration resistance of up to about 500 kPa. A similar limit value typically applies to extraction resistance.

The geometry of the penetrometer should be scaled according to the corresponding part of Clause 8. The penetrometer should be manufactured to a surface roughness, *Ra*, of 0,4 µm ± 0,25 µm.

The penetrometer of the MCPT and the adjoining push rod should have the same diameter for at least 10 times the diameter above the cone tip. The diameter of the push rod for the MTPT and MBPT should be such that the minimum ratio of the projected area of respectively the T-bar and ball is 7:1, with no change in diameter of the adjoining push rod for a distance of 10 times the diameter of the rod.

The penetrometer of the MCPT can be designed to measure only resistance on the cone.

The load sensor for MCPT penetration resistance should be positioned within the penetrometer. The load sensor for the MTPT and MBPT should be positioned with the T-bar and ball or immediately above the T-bar and ball, respectively. A system with a load sensor that remains above the surface of the soil specimen can be used. In this case, correction for friction along the push rod should be applied..

The data acquisition system should be capable of recording the penetration depth and the signals from the sensor(s) of the penetrometer at a frequency such that the data point spacing is less than $0{,}5 \cdot D$. For the MCPT and MBPT, the symbol D represents the diameter of the cone and the ball, respectively. For the MTPT, the symbol D represents the diameter of the horizontal short bar.

B.2.15.8.4 Test procedures

The trajectory of the penetration test within the specimen should be such that the penetrometer maintains an edge-to-edge distance of at least $2 \cdot D$ from the confining surfaces of the sampler or sample liner. If multiple miniature penetration tests are performed within a single soil specimen, then the edge-to-edge test spacing for the later test should be at least $1 \cdot D_1 + 2 \cdot D_2$ where D_1 refers to D of any earlier test and D_2 refers to D of the later test.

The cone (T-bar, ball) resistance should be measured during penetration. If specified in the project specifications, the cone (T-bar, ball) resistance can be measured during retraction and during cyclic test phases. A cyclic test phase should comprise:

a) displacement-controlled cycling;

b) up to 10 full cycles, unless no further degradation in penetration resistance is evident over three or more cycles, in which case the test may be stopped sooner;

c) minimum up and down distances of $3 \cdot D$.

In some cases, important information can be obtained by carrying out load-controlled cyclic tests. The proximity of cycling close to the surface of the sample should be considered, where test results can differ significantly from results of deeper cycling. Water entrainment effects can be simulated by surface-breaking cyclic test phases, while maintaining free water at the surface of the soil specimen.

The rate of penetration, retraction and cycling should be nominally (20 ± 5) mm/s. In some cases, important information can be obtained by carrying out tests at other penetration rates. If such tests are carried out, the results should be clearly marked.

The readings from all sensors should be recorded at a data point spacing of less than $0{,}5 \cdot D$.

The standard uncertainty of penetration resistance and, where applicable, the extraction resistance should be better than 5 % of the measured value or 10 kPa, whichever is larger. In this regard, it is noted that miniature force sensors can be sensitive to transient temperature change and bending and torsion. The standard uncertainty of penetration depth should be better than 20 mm relative to the surface of the sample. The resolution of the measurements of penetration resistance should be better than one-third of the recommended standard uncertainty for penetration resistance.

B.2.15.8.5 Presentation of test results and reporting

The measured penetration resistance and, where applicable the extraction resistance, should be reported with penetration relative to the surface of the sample. The penetration resistance and the extraction resistance are defined as $q_{MCPT} = Q/A$, $q_{MTPT} = Q/A$ and $q_{MBPT} = Q/A$, where Q is the force in the direction of penetration and A is the projected area of the cone, T-bar or ball, respectively. The change in zero readings of the sensor for penetration resistance should be reported for each test. Such change can be due to the sensor itself and due to factors such as soil adhering to the penetrometer after extraction from the soil specimen and temperature variation throughout the specimen in case of testing of a freshly recovered box core sample.

The remoulded penetration resistance should be defined as the average penetration and extraction resistance over the last cycle.

The geometry of the soil specimen and the method of confinement should be reported. The trajectory of penetration and specimen orientation should be reported, if different from an equivalent in situ, vertically downward setting.

B.2.15.9 Remoulded undrained shear strength

The remoulded undrained strength, s_{ur}, of fine soils can be determined by several strength index test methods, including the fall cone (see B.2.15.2), miniature laboratory vane (see B.2.15.5), and UU triaxial compression test (depending on the consistency of the remoulded soil; see B.2.15.7), or in the ring shear apparatus (see B.5.3).

Remoulded shear strength can also be determined by ring shear testing (see B.5.3), MTPT (see B.2.15.8) and MBPT (see B.2.15.8).

For hand-remoulded tests, the soil should be remoulded at constant water content as described in 10.5.5, and the method of remoulding should be reported.

B.2.16 Soil sensitivity

Soil sensitivity, S_t, is the ratio of the undrained shear strength of the intact material, s_u, to that of the same material tested in a remoulded state, s_{ur}, i.e. $S_t = s_u/s_{ur}$.

Remoulded undrained shear strength used for soil sensitivity should be determined by the same strength index test method as used to determine the intact undrained shear strength.

B.2.17 Needle penetration resistance

NPR should be determined in accordance with ISRM (2014)[1]. NPR is a strength index for weak/weak rock. The test method requires a needle penetrometer and relies on recording the axial force and displacement of a needle pushed slowly into rock, typically to a penetration of 10 mm. The needle should have a length of 40 mm and a diameter ranging from 0,3 mm near the truncated cone tip to 0,1 mm. Test results are expressed as NPR, defined as the ratio of the axial force to penetration. NPR (in N/mm) can be correlated to the uni-axial compressive strength σ_c (MPa), as $\sigma_c = 0{,}402\text{NPR}^{0{,}929}$ (Ulusay and Erguler 2012[164]).

B.3 One-dimensional consolidation

B.3.1 General

One-dimensional consolidation tests, using a consolidometer, are performed to determine the one-dimensional stress-strain-flow characteristics of the soil, and to provide information related to the stress history of the soil. The loading programme and unloading/reloading loops should be carefully selected to meet test objectives. For low to medium OCR clays, the results can also be used to evaluate sample quality as described in 10.6.

Several types of consolidation tests can be performed, such as the IL test and the controlled rate of strain test typically referred to as the CRS test.

Consolidation tests should be performed in accordance with ISO 17892-5 or ASTM D2435 for IL test, or ASTM D4186 for CRS test.

Sample preparation methods are described in 10.5.

The specimen size (diameter and height) should follow the referenced standards. A specimen diameter should be applied as large as possible taking into account the relevant stress ranges to be tested in relation to the load capabilities of the apparatus in question.

A floating ring concept is generally recommended to reduce the friction between the soil and the ring.

NOTE 1 The CRS test can give a preconsolidation stress that is higher than that produced by an IL test that uses conventional 24 h load increments, due to the combination of rate effects in the CRS test and secondary compression during 24 h load increments in an IL test. CRS tests performed with an acceptable maximum base excess pore pressure, as specified in ASTM D4186, will generally produce values of σ'_p that are about 10 % greater than IL test results which are interpreted using end-of-primary-consolidation data [Mesri et al. (1994)][144].

Consolidation tests can be conducted with or without the use of filter paper between the specimen and porous discs. The procedure used should be documented in the description of test procedures. The choice of porous discs should match the soil to be tested. Porous discs should be clean and should be sufficiently porous and permeable to allow pore-fluid to pass without retardation.

Options for preparation of the porous discs include dry, moist (damp) or wet (saturated). The method used should be documented in the description of test procedures. The preparation method chosen should depend on the soil being tested and the need to prevent specimen swelling. For soils that readily absorb water (e.g. stiff clays) dry porous discs should be used and can be used for all other soils. Moist or wet porous discs can be used for specimens that have a low swelling potential. Moist discs are prepared by removing excess water from a saturated stone by placing them on a paper towel for several minutes and placing the stone into a dry base. Wet discs are prepared by placing a saturated stone into a water-filled consolidometer base and thereafter removing excess water with a paper towel.

When using dry porous discs, water potentially having the same ionic content as the specimen pore water should not be added to the porous discs before the vertical stress is high enough to prevent swelling.

The compartment and tubing below the bottom filter and above the top filter stone should remain dry until the stress required to prevent swelling has been reached. The stress at the time water is added to the porous discs/water bath should be documented. Evaporation from the specimen in the period before saturation should be effectively prevented.

For intact specimens, water can be added to the porous discs/water bath shortly after application of the seating load. The load should be immediately increased as required to prevent swelling.

Specimen evaporation should be prevented by using a water bath or tubing connected to the top cap and bottom pedestal filled with water to the same elevation as the specimen.

The occurrence of uncontrolled soil swelling should be prevented by adopting appropriate precautions during specimen re-consolidation, especially to high OC clays in which swelling is of particular concern.

The measured vertical deformation in a test should be corrected for apparatus deformation, which should be determined by calibration using a metal disc in place of the specimen.

NOTE The correction for apparatus deformation is likely to be important only for very stiff soils.

B.3.2 Incremental loading oedometer tests

As a standard loading sequence, the load level should be doubled at each load step, i.e. load increment ratio $\Delta P/P$ = 1,0 where ΔP is the change in stress for an increment and P is the current stress applied to the specimen. If determination of the preconsolidation stress, σ'_p, is important, the stress increments should be smaller (e.g. $\Delta P/P = 0{,}5$) for stresses around the expected σ'_p. A minimum of three subsequent load increments should be used after having passed/reached the estimated value of σ'_p. Small load increments often make it not possible to determine the coefficient of consolidation (c_v) using common graphical construction procedures such as the Taylor (1948)[160] square-root time method or the Casagrande (1938)[111][119] log time method.

Any deviation from this procedure should be given in the project specifications. The equipment should allow for multiple unload-reload cycles.

A doubling of the load should be used for loading and reloading except as noted above, whereas for unloading the stress can be a quarter of the stress of the previous consolidation stress. A minimum of 2 load steps should be applied for the unloading branch.

The load duration should allow the specimen, at a minimum, to reach the end of primary consolidation at every load step, as determined by the Taylor square-root time method and also described in ASTM D2435. If the coefficient of secondary compression is required for specific stress levels, then the load increment corresponding to those stress levels should be maintained for a minimum of one log cycle of time beyond the end of primary consolidation.

B.3.3 Continuous loading oedometer tests

There are several types of continuous loading test. Special considerations for the CRS test are given in this subclause. For other types of continuous loading test, detailed information on the equipment and the test procedures should be documented and reported.

For standards that require application of back-pressure to saturate the specimen and the base pressure measurement system (e.g. ASTM D4186), the CRS equipment should allow for application and control of this back-pressure. The CRS equipment should be capable of loading/unloading the soil specimen at a prescribed constant rate of vertical deformation (and thus strain). The soil specimen should be allowed to drain freely from the top only during CRS loading/unloading. The pore pressure should be measured at the bottom of the specimen through a stiff pressure-measuring system. As a check, the volume change of this device, when fully saturated, should not exceed 2 mm^3 when the pressure is increased from 70 kPa to 100 kPa.

The loading procedure should start with a vertical stress $\leq 0{,}25\ \sigma'_{v0}$, where σ'_{v0} is the in situ vertical effective overburden stress. The load should thereafter be applied at a constant rate of strain such that the pore pressure measured at the undrained bottom of the specimen is within 3 % to 15 % of the applied total vertical stress range during the loading phase of the test.

NOTE ASTM D4186 suggests that a good starting value for the strain rate would be 10 % per hour for high plasticity silts, 1 % per hour for low plasticity clays and 0,1 % per hour for high plasticity clays, and that an appropriate adjustment is to be made if needed during testing to keep the base pore pressure within 3 % to 15 % of the applied total vertical stress at all loading stages of the test. It is intended to adjust the strain rate as little as possible. It is thus recommended to undertake trials to determine the appropriate strain rate.

If an unload-reload cycle is required, the initial stress level of this cycle and the percent unload stress to be applied should be given in the project specifications. If an unload-reload cycle is conducted, the specimen should be allowed to sit at constant stress prior to commencement of the unload portion of the loop and, once again, prior to commencement of the reload part of the loop. The resting period should at least be long enough to allow for the base pore pressure to return to equilibrium.

B.3.4 Dismounting the specimen

At the end of the test the pressure should be reduced to zero. The specimen should be carefully removed from the oedometer cell and should be weighed. A water content determination should be made on the specimen.

The specimen should be split open and a soil description should be performed. Any observed intrusions and/or structure should be highlighted (e.g. slickensided, platey, blocky). A photograph of the tested specimen can be useful as well.

B.3.5 Determination of horizontal stress

Tests conducted using a specially instrumented consolidometer can be used to measure the horizontal stress during a consolidation test for assessment of the coefficient of lateral earth pressure at rest, K_0. Such tests should be performed using the same procedures as a regular consolidation test. Description of the consolidation cell with the horizontal stress measurement system and dimensions of the soil specimen should be documented and reported.

The value of K_0 can also be estimated by conducting K_0 consolidation for a consolidated triaxial test as described in B.4.5.2.

Due to sampling stress relief, relative to the in situ vertical effective stress, the laboratory determined value of horizontal stress is typically less than the in situ value during laboratory reconsolidation to the in situ vertical effective stress [e.g. Dyvik et al. (1985)[122] or Mesri and Hayat (1993)[143]. Therefore the vertical stress in the test should be increased well beyond the preconsolidation stress to obtain more reliable measurements of horizontal stress. This allows determination of the normally consolidated value of K_0 (i.e. with OCR = 1), and the subsequent use of one or more unload-reload cycles allows determination of K_0 versus OCR for mechanical unloading-reloading.

B.3.6 Presentation of results

Reported data should include all types of corrections, such as apparatus compressibility and friction in the load system. The form of presentation of results required should be given in the project specifications. This form can include plots of:

— vertical strain (and/or void ratio) versus vertical effective stress;

— constrained modulus versus vertical effective stress;

— coefficient of permeability versus vertical strain (and/or void ratio);

— coefficient of consolidation versus vertical effective stress;

— ratio of excess pore pressure to total vertical stress versus vertical effective stress (CRS test);

— vertical strain versus time [for individual increments of IL test];

— test duration;

— pore pressure versus time (CRS test).

The vertical effective stress and/or the coefficient of permeability should be reported in both linear and semi-logarithmic scales. The linear plots should have sufficient resolution at low stresses for accurate interpretation.

If the tabulated results include an estimate of the preconsolidation stress, the method(s) used to determine it should be reported.

B.4 Consolidated triaxial tests

B.4.1 General

Consolidated triaxial tests are performed to provide shear strength characteristics and stress-strain-strength relationships of the soil. For undrained tests with pore pressure measurements, dilatancy parameters are also provided. For low to medium OCR clays, triaxial tests with anisotropic consolidation to the estimated in situ effective stress state can also be used to evaluate sample quality as described in 10.6.

Major components of a typical triaxial test include:

— specimen preparation, including potential pre-conditioning/pre-shearing;

— saturation;

— consolidation;

— shearing.

Testing can involve either static or cyclic loading. Consolidation is typically isotropic, anisotropic or one-dimensional (i.e. area-constant K_0 consolidation). Shearing is usually either drained with measurement of volumetric change or undrained with measurement of shear-induced pore pressure.

Static triaxial tests should be performed in accordance with ISO 17892-9, ASTM D4767 or ASTM D7181.

Reported data should include all types of corrections. such as apparatus compressibility and friction in the load system.

The UCT and UU triaxial compression tests are described in B.2.15 under strength index tests.

B.4.2 Test apparatus

B.4.2.1 Triaxial cell

The sealing bushing and piston guide of the triaxial cell should be designed such that the piston runs smoothly and maintains alignment. If the axial force is measured outside the triaxial cell, the piston passing through the top of the cell and its sealing bushing should be designed so that the friction between them does not exceed 0,1 % of the axial force at failure.

For extension and cyclic tests, the piston connection to the top cap should be able to withstand tensile forces, and should be designed to give a low measurement error of deformation.

B.4.2.2 Rubber membrane

The thickness and material properties of the rubber membrane should be such that the calculated correction to the axial and radial stresses due to membrane stiffness in reference to the shear stress at failure (τ_f) is lower than:

— 15 % for τ_f < 12,5 kPa;

— 10 % for 12,5 kPa ≤ τ_f ≤ 25 kPa;

— 5 % for τ_f > 25 kPa.

The diameter of the unstretched membrane should be between 90 % and 95 % of the initial specimen diameter. Each membrane should be checked for leakage before use.

Dry membranes tend to absorb water; membranes should be soaked in water for at least 24 h before use.

B.4.2.3 Porous discs and filter paper

Porous discs should have plane and smooth surfaces. Their compressibility should be negligible compared to that of the specimen. The choice of porous discs should match the soil to be tested. Porous discs should be clean and should be sufficiently porous and permeable to allow pore-fluid to pass without retardation. The porous discs should have a coefficient of permeability in the range 5 m/s to 10 m/s. Cleaning and checks should be made to ensure adequate permeability of the porous discs.

Tests can be conducted with or without the use of filter paper between the specimen and the porous discs. Filter paper for lateral drainage is generally mounted either vertically around the specimen or in a spiral around the specimen. In cases where the filter paper is mounted vertically around the specimen, a correction to the vertical force acting on the specimen should be applied for the resistance of the filter paper. Whether filter paper is used or not should be documented in the description of test procedures.

Options for preparation of the porous discs include dry, moist (damp) or wet (saturated). The method used should be documented in the description of test procedures. The preparation method chosen should depend on the soil being tested and the need to prevent specimen swelling. For soils that readily absorb water (e.g. stiff clays) dry porous discs should be used and they can be used for all other soils. Moist or wet porous discs can be used for specimens that have a low swelling potential. Moist discs are prepared by removing excess water from a saturated stone by placing them on a paper towel for several minutes and placing the stone into a dry base. Wet discs are prepared by placing a saturated stone into a water-filled triaxial base and thereafter removing excess water with a paper towel.

When using dry porous discs, water potentially having the same ionic content as the specimen pore water, should not be added to the porous discs before the vertical and lateral stresses are high enough to prevent swelling. The compartment and tubing below the bottom porous disc and above the top porous disc should be dry until the stresses required to prevent swelling have been reached. The stresses at the time water is added to the porous discs should be documented. Evaporation from the specimen in the period before saturation should be effectively prevented.

The occurrence of uncontrolled soil swelling should be prevented by adopting appropriate precautions during specimen consolidation, especially to high OC clays in which swelling is of particular concern.

B.4.2.4 Maintaining constant fluid pressure

The device for keeping the cell and the pore pressures constant during consolidation should be accurate enough to keep the required difference between cell and pore pressures (the radial effective stress) constant to a standard uncertainty of less than 2 % of this difference or 0,7 kPa whichever is higher.

B.4.2.5 Load frame

The load frame should be able to provide a range of rates of vertical strain as required for the test (drained and undrained). The actual rate applied should not fluctuate more than 10 %.

NOTE 1 Frames with a load capacity in the range 10 kN – 50 kN, which are able to advance the piston with displacement rates varying from about 0,000 5 mm to about 2 mm per minute with a minimum of ten different constant rates have been found to be sufficient for most monotonic testing.

NOTE 2 Testing of highly dilative soils (e.g. very dense sand) can involve large reaction forces, for which the triaxial testing apparatus is to be sufficiently stiff to withstand.

For cyclic loading, the loading equipment required should be for load-controlled cyclic triaxial tests. Sinusoidal load-wave forms should be imposed on the soil specimens. The use of other types of equipment and load-wave forms should be documented. Typical frequency values required for load controlled cyclic triaxial tests vary from the order of about 0,1 Hz (sea waves) to about 1 Hz (seismic loading).

The specimen should be subjected to cyclic loading that, depending on the initial shear stress and cyclic amplitude, can induce stress reversals in the form of alternating cycles of vertical compression and extension loads about some specified average stress state, keeping the cell pressure constant to a standard uncertainty of less than 2 % or 0,7 kPa whichever is higher.

The loading rod-to-piston connection should be:

a) easy to install;

b) prevent any slip under load reversal or vibration;

c) prevent application of torsion to the specimen;

d) compensate for eccentricity between line of action of the loading equipment and the piston.

B.4.2.6 Transducers

B.4.2.6.1 Force

The axial force applied to the specimen by the piston through the top of the triaxial cell should be measured with a standard uncertainty of less than 1 % of the peak force at failure or 2 N whichever is higher. If the force is measured with a device installed inside the triaxial cell, the device should be insensitive to horizontal forces and eccentricities in the axial force and should not be influenced by the magnitude of cell pressure.

If an external force transducer is used, the piston friction and piston uplift force correction should be accounted for. This can be avoided by using an internal force transducer.

B.4.2.6.2 Pressure

The pore pressure measurement system should be as rigid as possible. Formula (B.1) should be applied for static tests and Formula (B.2) should be applied for cyclic test:

$$\frac{\Delta V_{\mathrm{ms}}}{(V\times\Delta u)}\leq 0{,}5\cdot 10^{-6} \qquad \text{(B.1)}$$

$$\frac{\Delta V_{\mathrm{ms}}}{(V\times\Delta u)}\leq 0{,}1\cdot 10^{-6} \qquad \text{(B.2)}$$

where

ΔV_{ms} is the change in volume of the pore pressure measurement system due to pore pressure change, expressed in m^3;

Δu is the pore pressure change, expressed in kN/m^2;

V is the total volume of the specimen, expressed in m^3.

NOTE In testing clays, the use of a local (on-specimen) pore pressure measurement system (e.g. Hight (1982) [124]) minimizes partial pore pressure equalization effects during undrained shear tests and provides a more accurate measurement of specimen pore pressure.

B.4.2.6.3 Deformation

Axial displacement of the piston should be measured with a standard uncertainty better than 0,02 % of the initial specimen height. Possible measurement errors of displacement readings due to cell pressure change, apparatus deformation, use of membranes and filter papers should be accounted for. The apparatus should be sufficiently stiff for the specific soils to be tested.

NOTE Due to factors ,such as bedding errors, specimen tilting and apparatus deformation [Jardine et al. (1984)[130], Baldi et al. (1988)][100], conventional external measurement of displacement is often not representative of specimen deformation below axial strain levels of about 0,5 %, particularly for stiff soils [Clayton and Hight (2007)][115].

Local (internal or on-specimen) axial displacement systems provide more accurate measurement of the small-strain deformation with the potential to reliably measure deformation down to 0,001 % of the initial specimen height. If the project specifications require accurate measurement of small-strain deformation behaviour, then a local displacement measurement system should be used. Clayton and Hight (2007)[115] lists various systems that can be used for local measurement of axial displacement.

For cyclic tests, the deformation transducers should have a peak-to-peak amplitude strain range of at least 30 % of the sample height prior to the start of the cyclic test.

The volumes of water and air going into or out of the specimen should be measured with a standard uncertainty of better than 0,04 % of the total volume of the specimen.