

BS EN ISO 19901-4:2016



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Petroleum and natural gas industries — Specific requirements for offshore structures

Part 4: Geotechnical and foundation design considerations (ISO 19901-4:2016)

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National foreword

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The UK participation in its preparation was entrusted to Technical Committee B/525/12, Design of offshore structures.

A list of organizations represented on this committee can be obtained on request to its secretary.

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English Version

**Petroleum and natural gas industries - Specific
requirements for offshore structures - Part 4: Geotechnical
and foundation design considerations (ISO 19901-4:2016)**

Industries du pétrole et du gaz naturel - Exigences
spécifiques relatives aux structures en mer - Partie 4:
Bases conceptuelles des fondations (ISO 19901-
4:2016)

Erdöl- und Erdgasindustrie - Besondere
Anforderungen an Offshore-Bauwerke - Teil 4:
Geotechnische und Fundament-Auslegungsmerkmale
(ISO 19901-4:2016)

This European Standard was approved by CEN on 2 July 2016.

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COMITÉ EUROPÉEN DE NORMALISATION
EUROPÄISCHES KOMITEE FÜR NORMUNG

CEN-CENELEC Management Centre: Avenue Marnix 17, B-1000 Brussels

European foreword

This document (EN ISO 19901-4:2016) has been prepared by Technical Committee ISO/TC 67 "Materials, equipment and offshore structures for petroleum, petrochemical and natural gas industries" in collaboration with Technical Committee CEN/TC 12 "Materials, equipment and offshore structures for petroleum, petrochemical and natural gas industries" the secretariat of which is held by NEN

This European Standard shall be given the status of a national standard, either by publication of an identical text or by endorsement, at the latest by February 2017, and conflicting national standards shall be withdrawn at the latest by February 2017.

Attention is drawn to the possibility that some of the elements of this document may be the subject of patent rights. CEN [and/or CENELEC] shall not be held responsible for identifying any or all such patent rights.

This document supersedes EN ISO 19901-4:2003.

According to the CEN-CENELEC Internal Regulations, the national standards organizations of the following countries are bound to implement this European Standard: Austria, Belgium, Bulgaria, Croatia, Cyprus, Czech Republic, Denmark, Estonia, Finland, Former Yugoslav Republic of Macedonia, France, Germany, Greece, Hungary, Iceland, Ireland, Italy, Latvia, Lithuania, Luxembourg, Malta, Netherlands, Norway, Poland, Portugal, Romania, Slovakia, Slovenia, Spain, Sweden, Switzerland, Turkey and the United Kingdom.

Endorsement notice

The text of ISO 19901-4:2016 has been approved by CEN as EN ISO 19901-4:2016 without any modification.

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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).

Attention is drawn to the possibility that some of the elements of this document may be the subject of patent rights. ISO shall not be held responsible for identifying any or all such patent rights. Details of any patent rights identified during the development of the document will be in the Introduction and/or on the ISO list of patent declarations received (see www.iso.org/patents).

Any trade name used in this document is information given for the convenience of users and does not constitute an endorsement.

For an explanation on 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 the following URL: www.iso.org/iso/foreword.html.

The committee responsible for this document is ISO/TC 67, *Materials, equipment and offshore structures for the petroleum, petrochemical and natural gas industries*, Subcommittee SC 7, *Offshore structures*.

This second edition cancels and replaces the first edition (ISO 19901-4:2003), which has been technically revised.

ISO 19901 consists of the following parts, under the general title *Petroleum and natural gas industries — Specific requirements for offshore structures*:

- *Part 1: Metocean design and operating considerations*
- *Part 2: Seismic design procedures and criteria*
- *Part 3: Topsides structure*
- *Part 4: Geotechnical and foundation design considerations*
- *Part 5: Weight control during engineering and construction*
- *Part 6: Marine operations*
- *Part 7: Stationkeeping systems for floating offshore structures and mobile offshore units*
- *Part 8: Marine soil investigations*

The following part is under preparation:

- *Part 9: Structural integrity management*

ISO 19901 is one of a series of standards for offshore structures. The full series consists of the following International Standards which are relevant to offshore structures for the petroleum and natural gas industries:

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

- ISO 19901 (all parts), *Petroleum and natural gas industries — Specific requirements for offshore structures*
- ISO 19902, *Petroleum and natural gas industries — Fixed steel offshore structures*
- ISO 19903, *Petroleum and natural gas industries — Fixed concrete offshore structures*
- ISO 19904, *Petroleum and natural gas industries — Floating offshore structures*
- ISO 19905-1, *Petroleum and natural gas industries — Site-specific assessment of mobile offshore units — Part 1: Jack-ups*
- ISO/TR 19905-2, *Petroleum and natural gas industries — Site-specific assessment of mobile offshore units — Part 2: Jack-ups commentary and detailed sample calculation*
- ISO 19905-3, *Petroleum and natural gas industries — Site specific assessment of mobile offshore units — Part 3: Floating units (under preparation)*
- ISO 19906, *Petroleum and natural gas industries — Arctic offshore structures*

Other ISO standards can have implications for the geotechnical design of foundations for offshore structures, in particular:

- ISO 13623 (all parts), *Petroleum and natural gas industries — Pipeline transportation systems*
- ISO 13628 (all parts), *Petroleum and natural gas industries — Design and operation of subsea production systems*

Introduction

The International Standards for offshore structures, ISO 19900 to 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 type of structure and the nature of the materials used.

It is important to recognize that structural integrity is an overall 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 or structural system. The implications involved in modifications, therefore, need to be considered in relation to the overall reliability of all offshore structural systems.

For foundations, some additional considerations apply. These include the time, frequency and rate at which actions are applied, the method of foundation installation, the properties of the surrounding soil, the overall behaviour of the seabed, effects from adjacent structures and the results of drilling into the seabed. All of these, and any other relevant information, need to be considered in relation to the overall reliability of the foundation.

These International Standards are intended to provide wide latitude in the choice of structural configurations, materials and techniques without hindering innovation. The design practice for the foundations of offshore structures has proved to be an innovative and evolving process over the years. This evolution is expected to continue and is encouraged. Therefore, circumstances can arise when the procedures described herein or in ISO 19900 to ISO 19906 (or elsewhere) are insufficient on their own to ensure that a safe and economical foundation design is achieved.

Seabed soils vary. Experience gained at one location is not necessarily applicable at another, and extra caution is necessary when dealing with unconventional soils or unfamiliar foundation concepts. Sound engineering judgment is therefore necessary in the use of this part of ISO 19901.

For an offshore structure, the action effects at the interface between the structure's subsystem and the foundation's subsystem(s) are internal forces, moments and deformations. When addressing the foundation's subsystem(s) in isolation, these internal forces, moments and deformations can be considered as actions on the foundation's subsystem(s) and this approach is followed in this part of ISO 19901.

Some background to and guidance on the use of this part of ISO 19901 is provided for information in [Annex A](#). Guidance on foundations in carbonate soils is provided for information in [A.6.4](#), but there is, as yet, insufficient knowledge and understanding of such soils to produce normative requirements.

In this part of ISO 19901, in accordance with the latest edition of the ISO/IEC Directives, Part 2, the following verbal forms are used:

- 'shall' and 'shall not' are used to indicate requirements strictly to be followed in order to comply with the document and from which no deviation is permitted;
- 'should' and 'should not' are used to indicate that among several possibilities one is recommended as particularly suitable, without mentioning or excluding others, or that a certain course of action is preferred but not necessarily required, or that (in the negative form) a certain possibility or course of action is deprecated but not prohibited;
- 'may' and 'need not' are used to indicate a course of action permissible within the limits of the document;
- 'can' and 'cannot' are used for statements of possibility and capability, whether material, physical or causal.

Petroleum and natural gas industries — Specific requirements for offshore structures —

Part 4: Geotechnical and foundation design considerations

1 Scope

This part of ISO 19901 contains provisions for those aspects of geoscience and foundation engineering that are applicable to a broad range of offshore structures, rather than to a particular structure type. Such aspects are:

- site and soil characterization;
- identification of hazards;
- design and installation of shallow foundations supported by the seabed;
- design and installation of pile foundations;
- soil-structure interaction for auxiliary structures, e.g. subsea production systems, risers and flowlines (guidance given in [A.10](#));
- design of anchors for the stationkeeping systems of floating structures (guidance given in [A.11](#)).

Particular requirements for marine soil investigations are detailed in ISO 19901-8.

Aspects of soil mechanics and foundation engineering that apply equally to offshore and onshore structures are not addressed. The user of this part of ISO 19901 is expected to be familiar with such aspects.

ISO 19901-4 outlines methods developed primarily for the design of shallow foundations with an embedded length (L) to diameter (D) ratio $L/D < 1$ ([Clause 7](#)) and relatively long and flexible pile foundations with $L/D > 10$ ([Clause 8](#)). This part of ISO 19901 does not apply to intermediate foundations with $1 < L/D < 10$. Such intermediate foundations, often known as 'caisson foundations', comprise either shallow foundations with skirts penetrating deeper into the seabed than the width of the foundation, or shorter, more rigid and larger diameter piles than those traditionally used for founding offshore structures. The design of such foundations can require specific analysis methods; it is important that any extrapolation from the design methods described in this part of ISO 19901 to intermediate foundations be treated with care and assessed by a geotechnical specialist.

2 Normative references

The following documents, in whole or in part, are normatively referenced in this document and are indispensable for its application. For dated references, only the edition cited applies. For undated references, the latest edition of the referenced document (including any amendments) applies.

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

ISO 19901-1, *Petroleum and natural gas industries — Specific requirements for offshore structures — Part 1: Metocean design and operating considerations*

ISO 19901-2, *Petroleum and natural gas industries — Specific requirements for offshore structures — Part 2: Seismic design procedures and criteria*

ISO 19901-3, *Petroleum and natural gas industries — Specific requirements for offshore structures — Part 3: Toppers structure*

ISO 19901-5, *Petroleum and natural gas industries — Specific requirements for offshore structures — Part 5: Weight control during engineering and construction*

ISO 19901-6, *Petroleum and natural gas industries — Specific requirements for offshore structures — Part 6: Marine operations*

ISO 19901-7:2013, *Petroleum and natural gas industries — Specific requirements for offshore structures — Part 7: Stationkeeping systems for floating offshore structures and mobile offshore units*

ISO 19901-8, *Petroleum and natural gas industries — Specific requirements for offshore structures — Part 8: Marine soil investigations*

ISO 19902, *Petroleum and natural gas industries — Fixed steel offshore structures*

ISO 19903, *Petroleum and natural gas industries — Fixed concrete offshore structures*

ISO 19905-1, *Petroleum and natural gas industries — Site-specific assessment of mobile offshore units — Part 1: Jack-ups*

ISO 19906, *Petroleum and natural gas industries — Arctic offshore structures*

ISO/TR 19905-2, *Petroleum and natural gas industries — Site-specific assessment of mobile offshore units — Part 2: Jack-ups commentary and detailed sample calculation*

3 Terms and definitions

For the purposes of this document, the terms and definitions given in ISO 19900, ISO 19901 (all parts) and the following apply.

3.1 action

external load applied to the structure (direct action) or an imposed deformation or acceleration (indirect action)

Note 1 to entry: An imposed deformation can be caused by fabrication tolerances, differential settlement, temperature change or moisture variation. An earthquake typically generates imposed accelerations.

[SOURCE: ISO 19900:2013, 3.1]

3.2 action factor

partial safety factor applied to a design action

3.3 basic variable

one of a specified set of variables representing physical quantities which characterize actions, environmental influences, geometric quantities, or material properties including soil properties

[SOURCE: ISO 19900:2013, 3.7]

3.4 characteristic value

value assigned to a basic variable associated with a prescribed probability of not being violated by unfavourable values during some reference period

Note 1 to entry: The characteristic value is the main representative value. In some design situations a variable can have two characteristic values, an upper and a lower value.

Note 2 to entry: For variable actions, the characteristic value corresponds to either of the following (see ISO 2394:2015, 2.2.30):

- an upper value with an intended probability of not being exceeded or a lower value with an intended probability of being achieved, during some specific reference period;
- a nominal value, which may be specified in cases where a statistical distribution is not known.

[SOURCE: ISO 19900:2013, 3.10]

3.5

design actions

combination of representative actions and partial safety factors representing a design situation for use in checking the acceptability of a design

3.6

design value

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

[SOURCE: ISO 19900:2013, 3.18]

3.7

drained condition

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

[SOURCE: ISO 19901-8:2014, 3.11]

3.8

effective foundation area

reduced foundation area having its geometric centre at the point where the resultant action vector intersects the foundation base level

3.9

limit state

state beyond which the structure no longer satisfies the relevant design criteria

[SOURCE: ISO 19900:2013, 3.2]

3.10

material factor

partial safety factor applied to the characteristic strength of the soil, the value of which reflects the uncertainty or variability of the material property

Note 1 to entry: See ISO 19900.

3.11

representative value

value assigned to a basic variable for verification of a limit state

[SOURCE: ISO 19900:2013, 3.38]

3.12

resistance

capacity of a component, or a cross-section of a component, to withstand action effects without failure

[SOURCE: ISO 19900:2013, 3.39]

3.13

resistance factor

partial safety factor applied to the characteristic capacity of a foundation, the value of which reflects the uncertainty or variability of the component resistance including those of material property

3.14

scour

removal of seabed soils caused by currents, waves and ice

[SOURCE: ISO 19900:2013, 3.43]

3.15

seabed

materials below the seafloor, whether of soils such as sand, silt or clay, cemented materials or of rock

Note 1 to entry: Offshore foundations are most commonly installed in soils, and the terminology in this part of ISO 19901 reflects this. However, the requirements equally apply to cemented seabed materials and rock. Thus, the term 'soil' does not exclude any other material at or below the seafloor.

3.16

seafloor

interface between the sea and the seabed

3.17

serviceability

ability of a structure or structural member to perform adequately for a normal use under all expected actions

[SOURCE: ISO 2394:2015, 2.1.32]

3.18

settlement

permanent downward movement of a structure as a result of its own weight and other actions

3.19

strength

mechanical property of a material indicating its ability to resist actions, usually given in units of stress

Note 1 to entry: See ISO 19902.

3.20

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

[SOURCE: ISO 19901-8:2014, 3.42]

4 Symbols and abbreviated terms

4.1 General

Commonly used symbols are listed in [4.2](#) to [4.5](#); other symbols are defined in the text following the applicable formula. It should be noted that symbols can have different meanings between formulae.

4.2 Symbols for shallow foundations design

A	actual (cross-sectional plan) foundation area
A'	effective foundation area depending on eccentricity of actions
A_h	vertical projected area of the foundation in the direction of sliding
A_p	projected area of skirt tip

A_s	side surface area of skirt embedded at a particular penetration depth
$A_{idealized}$	idealized rectangular foundation area, for irregular foundation shapes
b_c, b_q, b_γ	bearing capacity correction factors related to foundation base inclination
B	minimum lateral foundation dimension (also foundation width)
B'	minimum effective lateral foundation dimension (also foundation effective width)
C	compression index of soil over loading range considered
d_c, d_q, d_γ	bearing capacity correction factors related to foundation embedment depth
D	foundation diameter (for circular foundations)
D_b	depth below seafloor to foundation base level
e	eccentricity of action
e_0	initial void ratio of the soil
e_1	eccentricity of action in coordinate direction 1
e_2	eccentricity of action in coordinate direction 2
f	unit skin friction resistance along foundation skirts during installation
F	bearing capacity correction factor to account for undrained shear strength heterogeneity
g_c, g_q, g_γ	correction factors related to seafloor inclination
G	elastic shear modulus of soil
h	soil layer thickness
H	horizontal action
H_b	horizontal action on effective area component of the base
H_d	design value of resistance to pure sliding
ΔH_d	horizontal soil resistance due to active and passive earth pressures on foundation skirts
H_{ult}	ultimate horizontal capacity in yield surface design method
i_c, i_q, i_γ	bearing capacity correction factors related to foundation action inclination
K_c, K_q, K_γ	correction factors that account for inclined actions, foundation shape, depth of embedment, inclination of base, and inclination of the seafloor
K_p	coefficient of passive earth pressure
K_{rd}	drained horizontal soil reaction coefficient
K_{ru}	undrained horizontal soil reaction coefficient
L	maximum lateral foundation dimension (also foundation length)
L'	maximum effective lateral foundation dimension (also foundation effective length)
M	overturning moment

M_{ult}	moment capacity in yield surface design method
N_c	undrained bearing capacity factor, equal to 5,14
N_q, N_γ	drained bearing capacity factors, as a function of ϕ'
p'_{in}	<i>in situ</i> effective overburden stress at skirt tip level inside the skirts of a skirted foundation
p'_{out}	<i>in situ</i> effective overburden stress at skirt tip level outside the skirts of a skirted foundation
q	unit end bearing resistance on foundation skirt tip, during penetration
q_d	design value of vertical bearing resistance in the absence of horizontal actions
Q	vertical action
Q_f	skirt friction resistance
Q_p	end bearing resistance from skirt tips
Q_r	soil resistance during skirt penetration
Q_{ult}	vertical capacity in yield surface design method
R	radius of the base of a circular foundation
RP	reference point for action transfer
s_u	undrained shear strength
s_{u0}	undrained shear strength at foundation base level (skirt tip level for skirted foundations)
$s_{u,ave}$	average undrained shear strength from seafloor to foundation base level
$s_{u,2}$	equivalent undrained shear strength below foundation base
s_c, s_q, s_γ	bearing capacity correction factors related to foundation shape
T	torsional moment
u_Q, u_H	vertical and horizontal displacements at foundation base level
β	ground inclination angle in radians, in calculation of inclination factors
δ	interface friction angle between soil and foundation
$\Delta\sigma'_{v,z}$	increment of effective vertical stress in a given soil layer at the specified time due to the increment of vertical action applied to foundation
ϕ'	effective angle of internal friction angle of the soil for plane strain conditions
γ'	submerged unit weight of soil
γ_L	live load partial factor
γ_m	material factor
κ	rate of increase of undrained shear strength with depth
$\sigma'_{v0,z}$	effective overburden stress at level of a given soil layer
σ'_{v0}	<i>in situ</i> effective overburden stress at foundation base level (skirt tip level when skirts are used)

ν	Poisson's ratio of the soil
ν	foundation base inclination angle in radians, in calculation of inclination factors
θ_M, θ_T	displacements at foundation base level under overturning and torsion loading

4.3 Symbols for pile foundations design

A_{pile}	gross end area of pile, $A_{\text{pile}} = \frac{\pi \cdot D^2}{4}$
A_r	pile displacement ratio, $A_r = \frac{A_w}{A_{\text{pile}}} = 1 - \left(\frac{D_i}{D} \right)^2$
A_w	cross-sectional area of pile annulus, $A_w = \frac{\pi}{4} \cdot (D^2 - D_i^2)$
A_s	side surface area of pile in soil
C_1, C_2, C_3	dimensionless coefficients determined as function of ϕ' , for p - y curves for sand
D	pile outside diameter
D_i	pile inside diameter, $D_i = D - 2 \text{ WT}$
D_{50}	mean soil particle diameter
D_{CPT}	diameter of CPT tool, $D_{\text{CPT}} = 36 \text{ mm}$ for a standard cone penetrometer with a cone area of $1\,000 \text{ mm}^2$
D_r	relative density of sand, for CPT-based methods 1 and 4
E_s	initial modulus of subgrade reaction
f	unit skin friction
$f(z)$	unit skin friction at depth z
$f_c(z)$	unit skin friction in compression at depth z
$f_p(z)$	unit skin friction between sand soil plug and inner pile wall, for CPT-based method 4
$f_t(z)$	unit skin friction in tension at depth z
f_{lim}	limiting unit skin friction value
h	distance above pile tip = $L - z$
J	dimensionless empirical constant, for p - y curves for clay
k	initial modulus of subgrade reaction, for p - y curves for sand
K_0	coefficient of lateral earth pressure at rest
L	embedded length of pile below original seafloor
L_s	length of soil plug in sand layers

N_q	dimensionless bearing capacity factor
p	mobilized lateral capacity, for p - y curves
p_a	atmospheric pressure ($p_a = 100$ kPa)
$P_{d,e}$	design value of axial action on the pile, determined from a coupled linear structure and non-linear foundation model using the design actions for extreme conditions
$P_{d,p}$	design value of axial action on the pile, determined from a coupled linear structure and non-linear foundation model using the design actions for permanent and variable actions or the design axial action for operating situations
p_r	representative value of lateral capacity, for p - y curves, in unit of force per unit length of pile
p_{rd}	representative value of deep lateral capacity, for p - y curves, in unit of force per unit length of pile
p_{rs}	representative value of shallow lateral capacity, for p - y curves, in unit of force per unit length of pile
$p'_m(z)$	<i>in situ</i> effective mean stress at depth z
P_o	pile outer perimeter = πD
q	unit end bearing at pile tip
$q_c(z)$	CPT cone resistance at depth, z , in stress units
$q_{c,f}(z)$	reduced CPT cone resistance at depth, z , to account for general scour
$q_{c,av,1,5D}$	average value of $q_c(z)$ between $1,5 D$ above pile tip and $1,5 D$ below pile tip
$q_{c,tip}$	CPT cone resistance at pile tip
Q	mobilized end bearing capacity in Q - z curves
$Q_{f,c}$	skin friction capacity in compression
$Q_{f,t}$	skin friction capacity in tension
$Q_{f,i,clay}$	cumulative skin friction capacity of clay layers within soil plug, for CPT-based method 3
Q_{lim}	limiting unit end bearing value
Q_p	end bearing capacity
Q_r	representative value of axial pile capacity
$Q_{r,c}$	representative value of axial pile capacity in compression
$Q_{r,t}$	representative value of axial pile capacity in tension
s_u	undrained shear strength
$s_u(z)$	undrained shear strength at depth z
WT	pile wall thickness
t	mobilized skin friction for axial shear transfer t - z curves
t_{max}	maximum skin friction for axial shear transfer t - z curves

t_{res}	residual skin friction for t - z curves
y	lateral pile displacement, for p - y curves
z	depth below original seafloor
z	local pile axial displacement, for axial shear transfer t - z curves
z	axial pile tip displacement, for Q - z curves
z_R	depth below seafloor to bottom of reduced resistance zone, for p - y curves for uniform clays
z'	final depth below seafloor, after general scour
z_{peak}	axial pile displacement at which maximum soil-pile skin friction, t_{max} , is reached, for t - z curves
z_{res}	axial pile displacement at which residual soil-pile skin friction, t_{res} , is reached, for t - z curves
α	dimensionless skin friction factor, for cohesive soils
β	dimensionless skin friction factor, for cohesionless soils
δ_{cv}	constant volume friction angle at sand-pile wall interface
ε_c	strain at one-half maximum deviator stress, for p - y curves for soft clay
ϕ'	effective angle of internal friction of sand, for drained triaxial conditions
γ'	submerged soil unit weight
γ_{pile}	unit weight of pile (steel, concrete, etc.)
γ_{water}	unit weight of water
$\gamma_{\text{R,Pe}}$	partial resistance factor for extreme conditions
$\gamma_{\text{R,Pp}}$	partial resistance factor for permanent and variable actions for operating situations
Ψ	parameter to determine the dimensionless skin friction factor, for clays = $s_u(z) / \sigma'_{\text{vo}}(z)$ at depth z
$\sigma'_{\text{ho}}(z)$	<i>in situ</i> effective horizontal stress at depth z
$\sigma'_{\text{vo}}(z)$	<i>in situ</i> effective vertical stress at depth z
$\sigma'_{\text{vo,tip}}$	<i>in situ</i> effective vertical stress at pile tip
Δz_{GS}	global scour depth
Δz_{LS}	local scour depth
e	base natural logarithms approximately 2,718
$\ln$	natural logarithm (base e)

4.4 Symbols for soil-structure interaction for auxiliary subsea structures, risers and flowlines

D	flowline, or pipeline, diameter
f_c	dimensionless cyclic factor
f_t	dimensionless time factor
f_v	dimensionless velocity factor
$G_{\max}$	initial elastic (small strain) shear modulus of soil
H	lateral (horizontal) soil resistance
$K_{\max}$	maximum value of normalized secant stiffness on initial unloading or reloading
I_p	plasticity index of soil
k_v	secant stiffness of equivalent spring = $\Delta Q / \Delta z$
N	integrated normal contact force
N_c	dimensionless bearing capacity factor
$Q_{\max}$	maximum suction (uplift) force, per unit length of pipeline
Q_u	limiting penetration resistance, per unit length of pipeline
s_u	undrained shear strength
s_{uDSS}	undrained shear strength obtained in direct simple shear mode
s_{ur}	remoulded undrained shear strength
T	drained axial resistance per unit length of pipeline
V	vertical action on pipeline
z	depth to flowline, or pipeline, invert
ΔQ	change in vertical force, per unit length of pipeline
Δz	change in vertical displacement
Δz_b	uplift (break-out) displacement
δ	interface friction angle at soil–pipeline interface
μ	pipeline–soil friction coefficient
ζ	dimensionless enhancement factor
ζ_t	dimensionless time factor
ζ_v	dimensionless velocity factor
θ_D'	half-angle of pipeline–soil contact perimeter

4.5 Symbols for design of anchors for stationkeeping systems

a	acceleration of a gravity embedded anchor
A	fluke area of a drag anchor
A_{eff}	effective area of a plate anchor accounting for shape and projected area
A_{in}	plan view inside area of suction anchor pile where underpressure is applied during installation
A_{inside}	inside lateral area of suction anchor pile wall
A_p	projected area of a gravity embedded anchor/line
A_{tip}	tip cross-sectional area of an anchor pile
A_{wall}	sum of inside and outside wall areas of an anchor pile
B	fluke width of a drag anchor
C_D	drag coefficient of a gravity-embedded anchor/line
f	coefficient of friction between chain or wire rope and the seafloor
F_b	bearing resistance of a penetrating gravity-embedded anchor/line
F_{drag}	hydrodynamic drag force action on a gravity-embedded anchor/line
F_f	frictional resistance of a penetrating gravity-embedded anchor/line
F_{max}	ultimate holding capacity (UHC) of a plate anchor
FOS_{axial}	factor of safety with respect to axial loading of anchor
FOS_{combined}	factor of safety with respect to combined axial and lateral loading of anchor
FOS_{lateral}	factor of safety with respect to lateral loading of anchor
H	horizontal action component
H	holding capacity of drag anchor under horizontal action
L	fluke length of a drag anchor
L_{cw}	length of chain or wire rope in contact with the seafloor
m	mass of a gravity-embedded anchor
n	dimensionless holding capacity factor for a drag anchor
N_c	dimensionless bearing capacity factor
P_{cw}	holding capacity of mooring line chain or wire rope
Q_{tot}	total penetration resistance of an anchor pile
Q_{side}	resistance along the sides of an anchor pile
Q_{tip}	resistance at the tip of an anchor tip
S_e	soil strength strain rate factor

S_t	soil sensitivity
s_u	undrained shear strength at the point in question
$s_{u,AVE}$	average undrained shear strength within the failure zone at the design penetration depth, corrected for effects of cyclic loading
$s_{u,tip AVE}$	average of triaxial compression, triaxial extension, and DSS undrained shear strength at anchor tip penetration depth
$s_{u,DSS}$	undrained shear strength obtained from direct simple shear tests
t	time
t_r	time of anchor retrieval
v	free-fall velocity of a gravity-embedded anchor
V	vertical action component
W_s	submerged weight of a gravity-embedded anchor
W'	submerged weight of anchor
W'_{cw}	submerged unit weight of chain or wire rope
z	embedment or penetration depth
ΔU_{req}	required underpressure to embed a suction anchor pile
ΔU_{crit}	critical underpressure causing failure of soil plug inside a suction anchor pile
α_{ins}	friction factor during installation of a suction anchor pile or of a gravity-embedded anchor
γ'	submerged unit weight of soil
η	empirical reduction factor accounting for progressive failure of a plate anchor
ρ	fluid density
θ	angle of mooring line at anchor padeye attachment point (measured from horizontal)
θ_{axial}	angle of mooring line at anchor padeye attachment point (measured from horizontal) above which the anchor ultimate capacity is controlled by axial capacity
$\theta_{lateral}$	angle of mooring line at anchor padeye attachment point (measured from horizontal) below which the anchor ultimate capacity is controlled by lateral capacity

4.6 Abbreviated terms

BOP	blow-out preventer
CPT	cone penetration test
FEM	finite element method
FOS	factor of safety
PFD	partial factor design

REB	reverse end bearing
SCR	steel catenary riser
SLS	serviceability limit state
SRD	soil resistance to driving
TDP	touch-down point
UHC	ultimate holding capacity
ULS	ultimate limit state
VLA	vertically loaded anchor

5 General requirements

5.1 General

The design methodology inherent to the ISO 19900 set of International Standards is based on the partial factor design (PFD) approach with specified action factors and material factors (see ISO 19900:2013, Clause 9). Requirements regarding partial factors for actions and the combination of actions into design situations are given in the relevant International Standards for offshore structures, i.e. ISO 19900 to ISO 19906. However, there are some aspects of design for which the PFD design formulations have not been developed and for these, other approaches are used, e.g. use of global safety factors in ISO 19901-7 for the design of stationkeeping systems even if PFD design methods exist for the design of anchors (see References [A.11-10], [A.11-11] and [A.11-14]).

With the historical development of both ISO 19901-4 and API RP 2GEO,[3] the terms ‘characteristic value’ and ‘representative value’ used throughout this part of ISO 19901 are both meant to represent a value with a prescribed probability of not being exceeded by unfavourable values during some specific reference period. In some design situations, two characteristic values can be defined, an upper and a lower value. If characteristic values cannot be determined from statistical data or where a statistical distribution is not known, if appropriate data are not available, a nominal value can be specified. By using this approach in combination with specified action factors and material factors, it is intended that geotechnical and foundation design will result in an offshore structure with an appropriate level of reliability, in general agreement with the requirements of ISO 2394 and ISO 19900.

The material factor for soil can be expressed as the ratio of the characteristic value of the undrained shear strength to the shear stress mobilized for equilibrium, or as the ratio of the characteristic value of the tangent of the representative angle of internal friction to the tangent of the angle of internal friction mobilized for equilibrium. The material factor should not be lower than 1,25. It can be modified with regard to the combination of consequences of failure, the accuracy of the applied calculation method or the model uncertainty and the way in which the characteristic strength of the soil material was determined and expressed.

Consideration should also be given to what is recognized practice for the calculation procedures applied and the stability mechanisms analysed. It is advised that the material factor should be reviewed and possibly increased particularly when new types of structures are considered and/or soil conditions with no or little experience are encountered.

The PFD design methodology in this part of ISO 19901 involves the use of action factors and material factors which aim to result in comparable overall foundation reliability with that achieved from use of the working stress design (WSD) methodology in the American Petroleum Institute’s Recommended Practice API RP2GEO.[3] For shallow foundations, this is outlined in Reference [6], which highlights that broadly comparable foundation size should be obtained to resist a given set of representative values of actions from either method. Although the design recommendations are largely aligned in this part of ISO 19901 and Reference [3], one should be aware that in certain circumstances the PFD or WSD

methods can lead to differences in the design outcome. It is recommended to seek guidance from a geotechnical specialist.

The foundation shall be designed to resist static and dynamic (repetitive as well as transient) actions without causing excessive deformation of or vibrations in the structure. Special attention shall be given to the effects of repetitive and transient actions on the structural response, as well as on the strength of the supporting soils. The possibility of movement of the seabed shall be considered. Any actions resulting from such movements on foundation members shall be considered in the design. The potential for disturbance to foundation soils by conductor installation or shallow well drilling shall be assessed (see 9.12). Furthermore, the guidance herein does not necessarily apply to unconventional soils such as carbonate material (see A.6.4), volcanic sands or highly sensitive clays.

Geotechnical and foundation design activities shall be performed by competent personnel with the qualifications and experience necessary to meet the objectives of this part of ISO 19901.

5.2 Design cases and safety factors

The design cases that shall be considered with the corresponding values of partial action factors are given in:

- ISO 19902 for fixed steel offshore structures;
- ISO 19903 for fixed concrete offshore structures;
- ISO 19904 and ISO 19901-7 for floating offshore structures;
- ISO 19905-1 for jack-ups;
- ISO 19906 for arctic offshore structures.

The resistance factors applicable to the design of pile foundations are given in 8.1.1. The material factors applicable to the design of shallow foundations are given in 7.3.1 and 7.3.3. In assessing the stability of shallow foundations, the design value of resistance is computed by applying a material factor to the characteristic soil strength. This differs from the practice for design of piles, where a resistance factor is applied to the characteristic foundation capacity.

The specific requirements and design procedures and criteria under dynamic actions from earthquakes are given in ISO 19901-2.

5.3 Characteristic values of soil properties

This subclause provides generic principles and guidelines for selecting characteristic values of soil properties, in line with the partial factors format or partial factor design (PFD) approach. The term 'soil' is used as per ISO 2394 and ISO 19900. This term is equivalent to 'seabed' defined in 3.15 and in ISO 19901-8.

Estimation of characteristic values for soil properties should consider the following:

- the definition used for the characteristic value;
- the assumptions made in the calculation model;
- the amount and quality of site investigations and possible environmental factors, including insufficient data and imprecise knowledge;
- a priori knowledge such as geological information and physically credible values;
- the measurable physical quantities that correspond to, and are representative of, the population of the properties considered in the calculation model;

- the appropriate factors or transformation functions, to convert the properties obtained from laboratory and/or *in situ* tests or other methods to properties corresponding to the assumptions made in the calculation model;
- the measurement error, conversion factor uncertainty and statistical uncertainty;
- the spatial variability of soil within each stratum;
- other variance reduction by appropriate methods (if relevant and if enough data are available).

Additional guidance on characteristic values of soil properties is given in [A.5.3](#).

5.4 Testing and instrumentation

Where there is uncertainty regarding the behaviour of foundations, testing or instrumentation should be undertaken. For all types of offshore structures, it is the stakeholder's responsibility to consider the need for instrumentation or monitoring of performance during installation and operation.

Possible testing and instrumentation methods include the following:

a) loading tests and/or model tests

Loading tests, model tests or large-scale field tests should be performed where there is particular uncertainty in the foundation capacity and where safety and/or economy are of particular importance, for example where:

- the foundation configuration differs significantly from earlier configurations where operational experience exists;
- the soil conditions differ significantly from those where operational experience exists;
- new methods of installation or removal are envisaged;
- a high degree of uncertainty exists as to how the structure or its foundation will behave.

b) temporary instrumentation

The structure or its foundation should be fitted with temporary instrumentation where:

- the installation method presupposes the existence of measured data for control of the operation;
- an installation method is to be applied with which little or no experience has been gained.

c) permanent instrumentation

The structure or its foundation should be fitted with permanent instrumentation where:

- the safety or behaviour of the foundation is dependent on active operation;
- the foundation configuration, the soil conditions, or the actions differ substantially from those with which experience has been gained;
- there is a need for monitoring the whole foundation with regard to penetration, settlement, tilt, or other behaviour;
- the method of removal presupposes the existence of measured data for control of the operation.

6 Geotechnical data acquisition and identification of hazards

6.1 General

The determination of the values of geotechnical parameters, and the assessment of geological hazards and constraints result from an integrated study of the area using geophysical surveys, geology, marine soil investigation and geotechnical engineering. Geophysical surveys should be performed first so that a shallow geohazard assessment should be performed based on the geophysical survey result before commencing the marine soil investigation. Additional guidance is provided in ISO 19901-8.

Geophysical data are acquired to develop a geological model so as to better understand depositional and other processes and features of an area. The geophysical data are also used to help interpret the stratigraphy from geotechnical boreholes, to define lateral variability across a site, and to provide guidance on optimizing the location of the proposed facilities. Incorporation of geotechnical data into the geological model gives insight into the potential impact of geological conditions on man-made facilities, such as structures, pipelines, anchors and wellheads.

6.2 Shallow geophysical investigation

Shallow geophysical investigation can provide information about soil stratigraphy and evidence of geological features, such as slumps, scarps, irregular or rough topography, mud volcanoes, mud lumps, collapse features, sand waves, slides, faults, diapirs, erosional surfaces, gas bubbles in the sediments, gas seeps, buried channels, and lateral variations in stratum thicknesses. The areal extent of shallow soil layers can sometimes be mapped if good correspondence is established between the soil boring and *in situ* test information and the results from the seabed surveys.

The types of equipment for performing shallow geophysical investigation that should be considered comprise

- a) Echo sounders or swathe bathymetric systems (in which a series of sweeps of the bathymetric equipment are used) define water depths and seafloor morphology. On complex seafloors, swathe systems have the advantage of providing higher data density and better definition of variable topography. These bathymetric surveys should be applied with tide measurements and corrections.

3D seismic data acquired for exploration purposes also provide useful data for developing water-bottom (bathymetry) maps in deep water. These data should only be used for preliminary evaluations because the resolution could be of the order of a few metres depending on the variability of the topography.

- b) Sub-bottom profilers (tuned transducers) define structural features within the near-surface sediments.

NOTE 1 These systems can also provide data to develop water-bottom or seabed rendering maps.

- c) Side-scan sonar defines seafloor features and seafloor reflectivity.

NOTE 2 Backscatter measurements from some swathe systems can also provide morphological information.

- d) Seismic sources, such as boomers or minisparkers, can define the structure up to approximately 100 m below the seafloor. Single or tuned arrays of sparkers, air guns, water guns or sleeve-exploders can define structure to deeper depths and can tie in with deep seismic data from reservoir studies. Seismic source signals are received either with single channel analogue or multi-channel hydrophones. Digital processing of the recorded signals enhances the quality of the images recorded and removes extraneous noise and multiples from the recorded signals.

- e) Seabed refraction equipment provides information on the stratification of the top few metres of the seabed.

In addition to boreholes, shallow sampling of near surface sediments using drop, piston or grab sampling, or vibrocoring together with cone penetrometer tests (CPT) along geophysical tracklines can be useful for calibration of results and improved definition of the shallow geology.

Direct observation of the seafloor using a camera mounted on a remotely operated vehicle (ROV) or on an autonomous underwater vehicle (AUV), or using a manned submersible can also provide important confirmation or characterization of geological conditions.

6.3 Geological modelling and identification of hazards

6.3.1 General

The nature, magnitude, and return intervals of potential active geological processes should be evaluated by site investigation techniques. Judicious use of analytical modelling can provide input for determination of the effects of active geological processes on structures and foundations. Due to uncertainties associated with definition of these processes, a parametric approach to studies is also helpful in the development of design criteria.

A geological model is constructed by hypothesizing a depositional process. The geophysical data should be mapped within the context of the hypotheses made. Features within the same geological period should be mapped together. Features not associated with a particular process should be mapped separately. If necessary, the mapping strategy should be adjusted to fit the model until agreement exists between the data and the model. The results of the geological modelling phase should ideally allow the interpreter to discuss in a report how features have developed over time, in order to allow assessment of how the features can affect future man-made developments.

Some of the more familiar geological processes, events and conditions are discussed in [6.3.2](#) to [6.3.7](#).

6.3.2 Earthquakes

Areas are considered seismically active on the basis of the historical record of earthquake activity, both in frequency of occurrence and in magnitude, or on the basis of a tectonic review of the region. For more details, see ISO 19901-2.

Seismic considerations for such areas can include investigation of the subsurface soils for instability due to liquefaction, submarine slides triggered by earthquake activity, proximity of the site to seismogenic faults, the characteristics of the ground motions expected during the life of the structure, and the acceptable seismic risk for the type of operation intended. Structures in shallow water that might be subjected to tsunamis should be investigated for the effects of the resulting actions.

6.3.3 Fault planes

In some offshore areas, fault planes can extend to the seafloor with the potential for vertical and horizontal movement. Fault movement can occur as a result of tectonic activity, removal of fluids from deep reservoirs or long-term creep related to large-scale sedimentation or erosion. Siting of facilities in close proximity to fault planes intersecting the seafloor should be avoided, if possible.

If circumstances dictate siting structures near potentially active faults, the effect of future fault movement on the foundation should be considered. If these effects are shown to be detrimental, the magnitude and time scale of expected movement can be estimated on the basis of a geological study for use in the design of structures.

6.3.4 Seafloor instability

Movements of the seafloor can be caused by ocean wave pressures, earthquakes, soil self-weight, hydrates, shallow gas, faults, or other geological processes. Weak, underconsolidated sediments occurring in areas where wave pressures are significant at the seafloor are susceptible to wave-induced movement and can be unstable under very small slope angles. Earthquakes can induce failure of seafloor slopes that are otherwise stable under the existing soil self-weight and wave actions.

Rapid sedimentation (such as actively growing deltas), low soil strength, soil self-weight, and wave-induced pressures are generally controlling factors for the geological processes that continually move sediment downslope. Important design considerations under these conditions include the effects of large-scale movement of sediment (i.e. mud slides and slumps) in areas subjected to strong wave pressures, downslope creep movements in areas not directly affected by wave/seafloor interaction and the effects of sediment erosion and/or deposition on structure performance.

The scope of site investigations in areas of potential instability should consider identification of metastable geological features surrounding the site and definition of the soil engineering properties required for modelling and estimating seafloor movements.

Estimates of soil movement as a function of depth below the seafloor based on geotechnical analyses can be used to predict actions on structural members. Geological studies employing historical bathymetric data or age dating of sediments can be useful for quantifying deposition rates during the design life of the facility.

6.3.5 Scour and sediment mobility

Scour is the removal of seabed soils by currents, waves and ice. Such erosion can be due to a natural geological process or can be caused by structural components interrupting the natural flow regime above the seafloor.

From observations, seafloor variations can usually be characterized as some combination of the following:

a) local scour

Steep-sided scour pits around foundation components such as piles and pile groups.

b) global scour

Shallow scoured basins of large extent around a structure, possibly due to overall structure effects, multiple structure interaction, or wave-soil-structure interaction.

c) overall seabed movement of sand waves, ridges, and shoals that would also occur in the absence of a structure

Such movements can result in seafloor lowering or rising, or repeated cycles of these. The addition of man-made structures often changes the local sediment transport regime that can aggravate erosion, cause accumulation, or have no net effect.

Scour can result in removal of vertical and lateral support for foundations, causing undesirable settlements of shallow foundations and overstressing of foundation components. Where scour is a possibility, it shall be taken into account in design and/or its mitigation shall be considered [see [A.8.1.4.2.7](#) item g) and [A.8.5.6](#)].

6.3.6 Shallow gas

The presence of either biogenic or petrogenic gas in the pore water of shallow soils is an important consideration to the engineering of the foundation. *In situ* natural gas can be either gaseous (present as gas dissolved within the pore space or free gas) or bound with water to form a solid (known as hydrate). In addition to being a potential drilling hazard during both site investigation soil borings (see ISO 19901-8) and oil well drilling, the effects of shallow gas can be important to foundation engineering. The effect of dissolution and expansion of gas in recovered soil samples can affect laboratory test results and derivation of geotechnical parameter values.

6.3.7 Seabed subsidence

The nature of the soil conditions and the reservoir and extraction processes should be investigated to establish whether subsidence of the seabed is likely to occur during the field life. The potential for

seabed subsidence, where this is a possibility due to reservoir compaction, should be addressed by considering an increase in air gap. The magnitude of surface settlements will depend on the reservoir dimensions, compressibility of reservoir rocks and the anticipated pressure drop.

6.4 Carbonate soils

When performing site investigations in frontier areas or areas known or suspected to contain carbonate material, the investigation should include diagnostic methods to determine the existence of carbonate soils. Particularly in sands and silts that contain in excess of 15 % to 20 % carbonate material, foundation behaviour can be adversely affected and a carefully developed field and laboratory testing programme can be warranted (see [A.6.4](#) and ISO 19901-8).

7 Design of shallow foundations

7.1 General

Shallow foundations in the context of this part of ISO 19901 include foundations placed directly on the seabed without embedment and embedded foundations with a maximum embedment of half the shortest plan dimension, i.e. an embedment ratio of 0,5. The guidance provided can be applicable to foundations with embedment ratios up to 1,0 but specialist advice should be sought in these cases.

The formulae presented for evaluating the stability and serviceability (displacement) of shallow foundations are given and are based on solutions for simple soil profiles and idealized soil response (i.e. uniform or linearly increasing strength or stiffness with depth and fully drained or undrained soil response). The formulae should only be applied to conditions similar to that for which they were derived or for which they can be shown to be applicable. Additional formulae that can be used for design are provided in [A.7](#) together with discussion of limitations and alternative approaches.

This Clause considers verification of limit states governed by soil behaviour and excludes verification of structural integrity of the foundation.

The following is noted.

- It is primarily intended that these recommendations be applied to the design of temporary foundations, such as jacket mudmat foundations with shallow skirts used for temporary support during installation, under idealized conditions.
- The methods presented are intended for combinations of dead action, variable live action and environmental action (i.e. wind, wave and current); where these arise from transient or cyclic actions they are considered as quasi-static actions. Specialist advice shall be sought where more complex dynamic analysis is required, such as where the inertia of the structure or foundation soils is important (e.g. seismic loading).
- These recommendations can also be considered for foundations that are to be used for permanent support. This is applicable only where permanent foundations are subject to simple loading and soil conditions, for which the recommendations remain valid; or for situations where foundation failure would result in displacements and rotations that could be shown to have minimal environmental, safety and economic consequences.
- Alternative design approaches can be more applicable for design of shallow foundations used in more complicated and critical situations, such as complex seabed conditions or complex multi-directional cyclic actions (including cases in which cyclic uplift actions can largely offset the foundation dead weight). Advanced analysis techniques such as finite element modelling can also be used to confirm that design requirements are met for more complex situations. Ultimate limit state under uplift requires special attention with due regard to negative pore pressures (suction), adhesion, permeability of the soil, drainage paths, duration of the action and geometry of the foundation. Unless other failure modes are more critical, such as where separation of the foundation from the seabed occurs due to drainage, uplift resistance can be analysed as a reverse bearing mechanism applying the relevant formulae for general shear failure in compression presented in [A.6.4](#).

- For large concrete gravity base structures and mobile offshore units, the requirements in this Clause shall be supplemented and/or modified by requirements given in ISO 19903, ISO 19905-1 and ISO/TR 19905-2.

For many situations, it is advantageous to equip the foundation plate with vertical skirts around its periphery that penetrate into the seabed. If the foundation area is large, interior skirts forming skirt compartments under the plate foundation can be required. The presence of the skirts will in most cases (i) increase foundation stability, (ii) decrease foundation deformation, and (iii) reduce impact of seabed scour on the foundation.

For shallow foundations equipped with seabed penetrating skirts, the following is noted:

- care should be taken to appropriately transfer the seafloor actions to skirt tip level, as described in [A.7.2.3](#);
- when assessing stability based on loading at skirt tip level, it is important to ensure that the penetration is sufficient to avoid internal failure mechanisms forming within the skirted compartment, which might lower the overall capacity. Required penetration into the stronger soil to achieve this would typically be between 1/10 and 1/5 of the spacing between skirts, although this depends on the site specific soil profile and foundation action. A case-specific analysis is required to confirm an acceptable penetration;
- the extent of skirt penetration should be assessed. Additional geotechnical/structure assessment is required in the event full skirt penetration is unachievable and no measure is taken to ensure full penetration.

7.2 Principles

7.2.1 General principles

The following general principles shall be considered in assessing stability of shallow foundations:

- bearing failure constitutes any failure mode that could result in excessive combinations of vertical displacement, lateral displacement, or overturning rotation of the foundation; while pure sliding or torsional failure corresponds to a failure mode where the foundation translates or twists only in a horizontal plane;
- foundation stability can be analysed by limit equilibrium methods ensuring equilibrium between design actions and design resistance. The shape and location of the critical failure mechanism shall be determined, which will depend on the design actions, soil stratification and foundation geometry. Due consideration shall be given to the possibilities of excessive displacement and deformation of the foundation soil, and where these are critical, more complex analysis approaches are required;
- calculations using alternative methods of analysis shall include an explanation of any possible differences due to the method adopted;
- design actions shall be determined with due consideration of the design life of the foundation;
- seafloor gradient and/or installation tolerance shall be taken into account in design. Tolerable foundation tilt should be specified;
- undrained calculations shall be adopted where no drainage, and hence no dissipation of excess pore pressures, occurs during loading. This can occur as a result of the rate of loading or the impermeable nature of the soil. In contrast, drained calculations shall be adopted where no excess pore pressures arise during loading. Analysis of foundations subject to partial soil drainage during the loading event is complex, and specialist advice shall be sought in these cases;
- design can be based on serviceability (rather than stability) criteria, whereby the deformation of the foundation is assessed against allowable movement criteria. The appropriateness of adopting this approach will depend on the type of structure and its installation. The selection of appropriate

soil moduli (especially considering strain dependency) is essential in calculation of serviceability limit states;

- impact of structural openings shall be taken into account in design.

7.2.2 Sign conventions, nomenclature and action reference point

Vertical (Q), horizontal (H), overturning (M) and torsional (T) actions are centric and act at a reference point (RP) taken as the midpoint of the foundation at seafloor level, see [Figure 1](#). This is the point of structural action transfer. H and M can be co-planar.

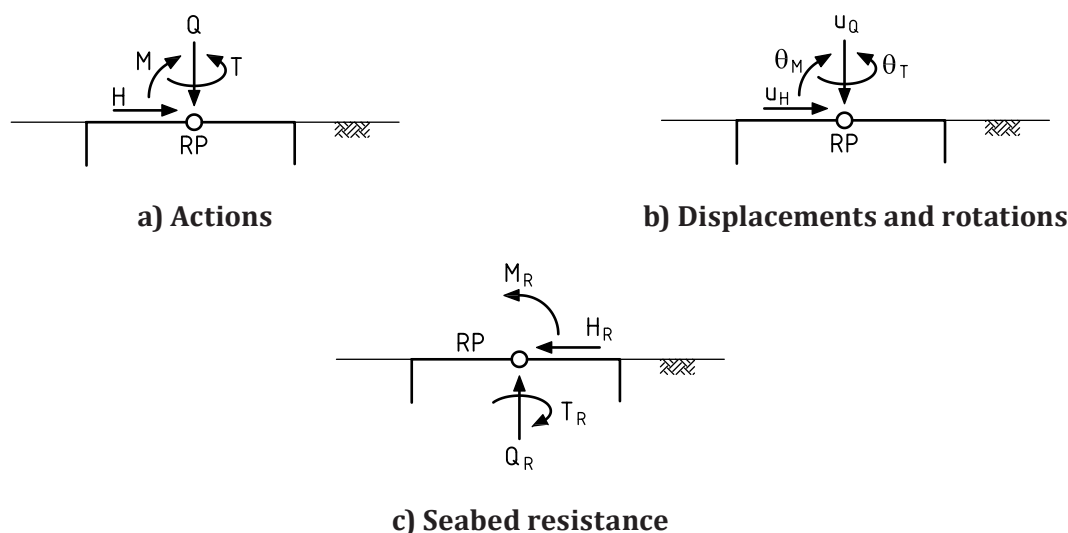


Figure 1 — Sign conventions, nomenclature and reference point for analysis of shallow foundation

7.2.3 Action transfer

For an embedded foundation action transfer from seafloor to base level (typically skirt tip level for a foundation equipped with skirts) should be calculated as described in [A.7.2.3](#).

7.2.4 Idealization of foundation area and the effective area concept

The methods outlined herein are based on the effective area concept. The effective foundation area is defined in [A.7.2.4](#), which also deals with idealization of the foundation area for use with limit equilibrium methods.

The effective area method might not be appropriate for use with highly compressible or layered soils, or for skirted foundations on soft soils and/or subject to high overturning moment. Specialist advice shall be sought in these conditions.

In drained conditions, the horizontal and vertical action effects shall be assumed as acting on the effective foundation area only. In undrained conditions, the action effects can be assumed as being distributed over a greater part than the effective area, due to mobilization of suction (relative to ambient pressure) between the underside of the foundation and seabed, potentially up to the total foundation area. In this case, there shall be documentation to show that the resulting stress distribution is possible and will not lead to new forms of failure with a lower safety level.

7.3 Acceptance criteria and design considerations

7.3.1 Action and material factors

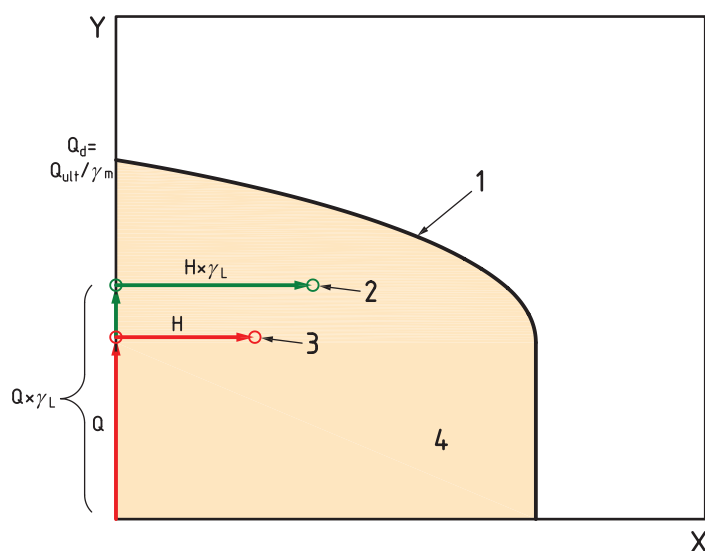
When calculating stability:

- the soil strength shall be determined using a material factor $\gamma_m = 1,25$ except as indicated below or in [A.7](#). Where geotechnical data are sparse or site conditions are particularly uncertain, or where high uncertainty exists in relation to potential failure mechanisms or methods of analysis, increasing the above material factor can be warranted. In accordance with ISO 19900, a material factor of 1,0 is permitted for accidental loading;
- partial action factors shall be determined based on guidance from the relevant standard of the ISO 19900- series. The weight of the soil, including that within the skirts, should normally be calculated with factors equal to unity (see [7.3.4.1](#)). In some situations, an action factor below or above 1,0 can be justified in order to ensure a sufficiently robust design;
- partial action factors and material factors shall be applied with consistency throughout the design process. In particular, note that the ratio between the resultant horizontal action and the resultant vertical action on the foundation will influence many of the design formulae provided in [A.7](#).

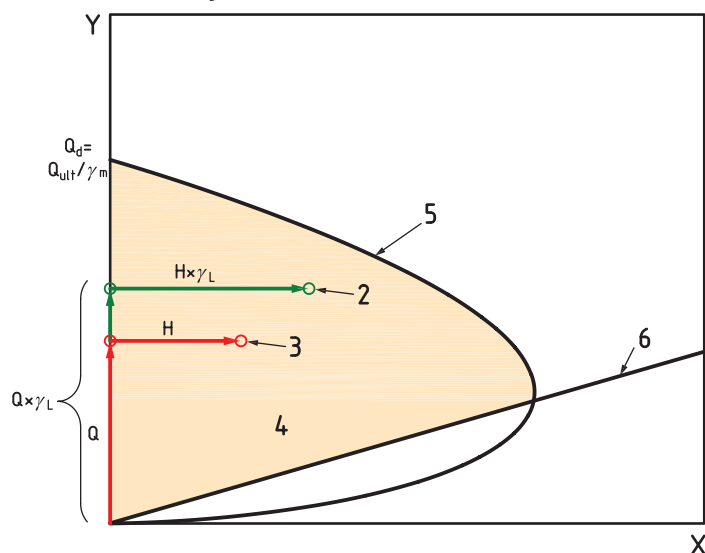
When calculating displacements as part of serviceability based design, all partial action and material factors should generally be set to unity. However, where foundation displacement can lead to unacceptable consequences or govern design, it can be appropriate to apply factors other than unity in order to ensure a sufficiently robust design. Specialist advice shall be sought in these cases.

7.3.2 Use in design

Numerical methods or the formulae presented in [7.4](#) can be used to develop a soil resistance envelope, incorporating the appropriate material factor. Typical examples of such failure envelopes are presented in [Figure 2](#). Once an envelope is derived, the design actions (incorporating appropriate partial action factors) can be transposed onto this envelope to ensure the safety requirements of this part of ISO 19901 are satisfied.



a) Undrained conditions



b) Drained conditions

Key

- | | |
|---|---|
| 1 envelope of design resistance under undrained bearing/sliding | 5 envelope of design resistance under drained bearing |
| 2 design value of action | 6 envelope of design resistance under drained sliding |
| 3 applied action | X horizontal action |
| 4 permissible design actions | Y vertical action |

Figure 2 — Soil resistance envelopes and definition of design actions for stability of shallow foundations under undrained and drained conditions

7.3.3 Special cases

For assessment of foundation stability during set-down on the seabed, stability can be assessed based on the applied vertical actions only. In this case, the ultimate limit state shall be calculated using a material factor of $\gamma_m = 1,5$. The increased material factor in this case leads to an increased margin

of safety against bearing failure during set-down and assists in settlement and/or embedment of the foundation into the seabed remaining modest.

For situations in which the geotechnical stability requirements are satisfied for all possible modes of failure, such as a foundation resting on very competent soil (e.g. a cemented seabed), design actions shall be checked to ensure that the foundation is not at risk from other non-geotechnical instabilities.

7.3.4 Additional design considerations

7.3.4.1 Adjusting for soil plug weight in skirted foundations

The general formulae presented in this part of ISO 19901 to calculate capacity assume there is no difference in the depth of soil inside and outside foundation skirts. However, in some cases

- the soil height above skirt tip level can be higher inside the skirt than outside the skirt, such as where significant scour has occurred, or
- the soil height above skirt tip level can be lower inside the skirt than outside the skirt, such as where the foundation (base plate and skirts) has penetrated deeper than the depth of the skirts.

In cases where a difference exists, the design vertical action should be adjusted by:

$$\Delta Q = (p'_{\text{in}} - p'_{\text{out}})A \quad (1)$$

where

ΔQ is the change in design vertical action to account for differences in vertical effective stress at skirt tip level. The factor for soil weight is generally equal to unity;

p'_{in} is the *in situ* effective overburden stress at skirt tip level inside the skirts (taken as $\gamma' z_{\text{in}}$, where γ' is the submerged unit weight of soil and z_{in} is the depth of soil inside the skirts);

p'_{out} is the *in situ* effective overburden stress at skirt tip level outside the skirts (taken as $\gamma' z_{\text{out}}$, where z_{out} is the depth of soil outside the skirts);

A is the actual cross-sectional plan foundation area.

Note that a similar method can be used for embedded shallow foundations without skirts. Care should be taken to ensure the design actions are adjusted appropriately in these cases.

7.3.4.2 Horizontal seabed resistance above foundation base level

Skirted or otherwise embedded shallow foundations can have increased resistance under pure sliding resulting from soil resistance above skirt tip level. If appropriate, this resistance can be used to offset horizontal actions transferred to the foundation base, such as when calculating inclination factors. All contributions to horizontal resistance from foundation members above foundation base level should be reduced using the material factor given in 7.3.1. Note that care should also be taken in conditions where installation disturbance or soil conditions can lead to lower resistance from the soil above skirt tip level (e.g. in conditions where soil tension cracking on the active side of the skirt can occur).

The effect of scouring shall be taken into account. In the absence of detailed scouring assessment, the passive resistance offered by the soil in front of the skirt shall be conservatively ignored for scour-prone seabeds.

The additional horizontal resistance, ΔH_d , that can be mobilized between seafloor and foundation base level can be approximately calculated from [Formulae \(2\) and \(3\)](#).

Total horizontal sliding resistance is given by $H_d + \Delta H_d$.

For undrained conditions, the additional horizontal resistance can be calculated using [Formula \(2\)](#):

$$\Delta H_d = K_{ru} \left(\frac{s_{u,ave}}{\gamma_m} \right) A_h \quad (2)$$

where

ΔH_d is the horizontal soil resistance due to active and passive earth pressures on foundation skirts;

K_{ru} is the undrained horizontal soil reaction coefficient (see [A.7.3.4.2](#));

$s_{u,ave}$ is the characteristic value of average undrained shear strength of soil between the seafloor and base level for linearly increasing isotropic undrained shear strength with depth;

γ_m is the material factor (see [7.3.1](#));

A_h is the vertical projected area of the foundation in the direction of sliding.

For drained conditions, the additional horizontal resistance can be calculated using:

$$\Delta H_d = K_{rd} (0,5 \gamma' D_b) A_h \quad (3)$$

where

ΔH_d is the additional horizontal resistance mobilized between the seafloor and foundation base level;

K_{rd} is the drained horizontal soil reaction coefficient, which includes the material factor (see [A.7.3.4.2](#));

γ' is the characteristic value of the average submerged unit weight of the soil over the depth of embedment;

D_b is the depth to base level;

A_h is the vertical projected area of the foundation in the direction of sliding.

7.3.4.3 Shallow foundations penetrating into soft soils

In soft soils (e.g. normally consolidated clays), foundations can penetrate into the seabed to the depth at which the soil bearing resistance is in equilibrium with the applied action, which implies no additional margin of safety. Allowable differential settlement of the structure will depend upon the type of structure and its installation, and should be the subject of a risk assessment. Adequate precautions should be taken to minimize differential settlements between foundations. Good practice should ensure that there is an acceptable limit to the penetration.

If the foundation is required to provide permanent support, normal practice is either to use skirts to transfer actions to deeper (more competent) soils, to increase foundation area, or to preload the structure to ensure that the foundation stability requirements under the design scenario are met.

7.3.4.4 Tensile stresses beneath foundation

Tensile stress (relative to ambient water pressure) beneath foundations that rest on the seafloor without embedment should be avoided because of the potential disturbance to the underlying seabed due to pumping scour, an erosional mechanism whereby rapid movement of water can lead to undermining of the foundation.

Skirted foundations (except those with perforated mudmats) can resist transient tension through generation of negative excess pore pressures between the confined soil plug and underside of the foundation top cover. Cyclic tensile stresses (relative to ambient water pressure) from waves with a

few seconds duration can often be demonstrated as acceptable in design; while longer duration tensile stresses can potentially be carried by skirted foundations on clays with low permeability. Specialized advice shall be sought in relation to reliance on tensile stresses beneath a foundation.

Uplift capacity can be analysed as a reverse bearing capacity if the permeability of the soil, drainage paths, duration of action and geometry of the foundation have been explicitly considered and shown not to jeopardize the negative excess pore pressures required to mobilize reverse end bearing.

7.3.4.5 Non-standard soils or soil profiles

The analysis methods outlined in this part of ISO 19901 was developed primarily for use in seabed conditions comprising uniform all drained (sand) or all undrained (clay) profiles. The methods require careful consideration when used in other soils, such as silts and other soils that can demonstrate partial drainage and/or complex soil behaviour. Specialist *in situ* and laboratory testing should be considered to determine the drainage and other characteristics of these materials for use in design.

Specialist geotechnical advice is required for foundation design in more complex soil profiles (e.g. layered), such as where 'punch-through' of foundations into underlying soils can occur or when the capacity of a shallow foundation can be reduced by the presence of thin layers of weaker soil at depth. Care is also required for the design of foundations on non-level or uneven seafloor.

7.3.4.6 Interaction with other structures

Potential influence of adjacent structures, such as jack-up spudcans or conductors, should be considered.

7.3.4.7 Multiple foundations

For foundations comprising several (connected) foundations, redistribution of loading between individual foundations will generally lead to an improvement in system performance, and should be considered. The interaction between closely spaced foundations can affect foundation capacity and shall be considered in the design. The interaction between multiple foundations shall be considered in calculations of foundation settlement and/or rotation.

7.3.4.8 Consideration of surrounding seabed conditions

Foundation design shall make allowance for ground conditions outside the base area where this can impact the assessment of foundation performance. Although relevant for all foundations, this is particularly relevant for foundations where failure can be associated with deep seated failure surfaces.

7.3.4.9 Carbonate soils

Carbonate soils require particular consideration. Generally, shallow foundations are suitable for use on carbonate soils, although any evaluation of such foundations should account for the differences that exist between such material and silica sands or clays.

Further discussion of carbonate soils is presented in [6.4](#) and [A.6.4](#) with particular reference to shallow foundations in [A.6.4.4.3](#).

7.3.5 Alternative method of design based on yield surfaces

It is generally accepted that the effective area method is conservative where large horizontal action and overturning moment act. An alternative method of design involves use of explicit yield functions to derive encompassing yield surfaces directly in vertical action, horizontal action, overturning moment and torsional space. Additional information on the yield surface approach is provided in [A.7.3.5](#).

7.3.6 Selection of soil parameter values for design

7.3.6.1 Shear strength used in stability analysis

Safe design of shallow foundations is strongly dependent on the quality of the site investigation performed and the methods used for determining strength and deformation properties of soils, both *in situ* and in the laboratory. ISO 19901-8 provides additional information on the requirements and quality of marine soil investigations.

Uncertainty in determining the characteristic value of shear strength can be significant. This can either prevent optimization of design or result in lower than intended safety margins. Such sources of uncertainty include:

- imprecise definition of the characteristic values of soil strength parameters used in the analysis methods;
- variability in strength measurements (which depends on the scope and content of the soil investigation, sampling disturbance, methods of testing, etc.);
- the strategies, methods, and procedures used for assessment of the characteristic value of strength, which vary from one designer or design environment to another;
- the amount of testing forming the basis for proposed characteristic values of shear strengths, statistical variability, and bias.

These uncertainties are relevant for both drained and undrained shear strengths and care should be taken in the selection of characteristic values of parameters (see [A.7.3.6.1](#)).

7.3.6.2 Parameters used in serviceability design

Parameter selection for serviceability design should consider the displacement condition being considered. For example, if calculating the largest likely ('upper bound') settlements, a consistent soil parameter set should be obtained for the most compressible soil likely at the site.

If elasticity based calculation methods are used, selection of equivalent linear elastic soil parameters should consider the strain levels that are induced in the seabed resulting from the applied actions.

7.4 Stability of shallow foundations

7.4.1 Assessment of bearing capacity

7.4.1.1 Undrained conditions (constant shear strength with depth)

[Formula \(4\)](#) is a general formula for determining the design unit bearing capacity for undrained conditions, with uniform isotropic undrained shear strength approximately constant with depth under the foundation:

$$q_d = N_c \frac{s_u}{\gamma_m} K_c \quad (4)$$

where

- q_d is the design vertical bearing resistance, and note that $Q_d = q_d A$;
- N_c is the undrained bearing capacity factor, equal to 5,14;
- s_u is the characteristic value of undrained shear strength of the soil;
- γ_m is the material factor (see 7.3);
- K_c is a correction factor, which accounts for inclined actions, foundation shape, depth of embedment, foundation base inclination and seafloor surface inclination.

Details for calculation of K_c are provided in A.7.

Formula (4) applies to situations with approximately constant undrained shear strength to a depth equal to at least 2/3 of the foundation width. For situations with low undrained shear strength immediately under the foundation and increasing with depth, very shallow bearing capacity failure mechanisms can occur. In this situation, the unit bearing capacity can be under-estimated using Formula (4) and guidance provided for linearly increasing shear strength should be used.

For a vertical centric action applied to a rough-based foundation at seafloor level where both the foundation base and seafloor are horizontal, Formula (4) is reduced as shown by Formulae (5) and (6) for strip, circular and square foundation geometries (for appropriately factored material shear strength).

— Infinitely long strip foundation

$$q_d = 5,14 \frac{s_u}{\gamma_m} \quad (5)$$

— Circular or square foundation

$$q_d = 6,05 \frac{s_u}{\gamma_m} \quad (6)$$

7.4.1.2 Undrained conditions (linearly increasing shear strength with depth)

Formula (7) is a general formula for determining design unit bearing capacity for undrained conditions with uniform isotropic undrained shear strength increasing approximately linearly with depth under the foundation.

$$q_d = F \left(N_c s_{u0} + \frac{\kappa B'}{4} \right) \frac{K_c}{\gamma_m} \quad (7)$$

where

- q_d is the design vertical bearing resistance, and note that $Q_d = q_d A$;
- F is a correction factor given as function of $\kappa B'/s_{u0}$;
- N_c is the undrained bearing capacity factor, equal to 5,14;
- s_{u0} is the characteristic value of undrained shear strength of the soil at foundation base level (skirt tip level for skirted foundations);
- κ is the rate of increase of the characteristic value of undrained shear strength with depth;

B' is the minimum effective lateral foundation dimension (see 7.2.4);

K_c is a correction factor, which accounts for inclined actions, foundation shape, depth of embedment, foundation base inclination and seafloor surface inclination;

γ_m is the material factor (see 7.3).

Details for calculation of F and K_c are provided in A.7.

7.4.1.3 Drained conditions

Formula (8) is a general formula for determining design vertical bearing capacity for drained conditions.

$$q_d = 0,5\gamma' B' N_\gamma K_\gamma + \sigma'_{v0} (N_q - 1) K_q \quad (8)$$

where

q_d is the design vertical bearing resistance in the absence of horizontal actions, and note that $Q_d = q_d A$;

N_γ, N_q are drained bearing capacity factors, as a function of ϕ' ;

K_γ, K_q are correction factors that account for inclined actions, foundation shape, depth of embedment, inclination of base, and inclination of the seafloor;

γ' is the characteristic value of submerged unit weight of soil;

σ'_{v0} is the *in situ* effective overburden stress at foundation base level (skirt tip level when skirts are used, taking care to correct this appropriately as per 7.3.4.1);

B' is the minimum effective lateral foundation dimension (see 7.2.4).

Complete descriptions of the K factors and values of N_q and N_γ as a function of the effective angle of internal friction ϕ' , are given in A.7.

Formula (8) has deliberately omitted any component due to an effective cohesion, c' , and accompanying bearing capacity factor, N_c . This is mainly because the occasions when it might be appropriate to include a component for bearing capacity due to a presumed effective cohesion are extremely rare, and specialist geotechnical advice shall be sought before any such inclusion. More advice is given in A.7.

For a vertical central action applied to a foundation at seafloor level where both the foundation base and seafloor are horizontal, Formula (8) is reduced as follows for the following foundation shapes:

— Infinitely long strip foundation

$$q_d = 0,5\gamma' B N_\gamma \quad (9)$$

— Circular or square foundation

$$q_d = 0,3\gamma' B N_\gamma \quad (10)$$

7.4.2 Assessment of sliding capacity

7.4.2.1 General

When assessing sliding capacity, due consideration shall be given during site investigation and subsequent interpretation to the possible occurrence of discrete layers of low strength soil, which can provide a preferential failure surface.

When stability has been established using the formulae in 7.4.1, the maximum horizontal capacity should be limited to that determined for the condition of pure sliding, as defined by Formulae (11) and (12).

7.4.2.2 Undrained conditions

Formula (11) can be used for determining undrained sliding capacity at the base of a rough foundation (skirt tip level for skirted foundations with appropriate skirt depth to spacing ratio):

$$H_d = \left(\frac{s_{u0}}{\gamma_m} \right) A \quad (11)$$

where

H_d is the design resistance for pure sliding;

s_{u0} is the characteristic value of undrained shear strength at foundation base level (skirt tip level for skirted foundations);

γ_m is the material factor (see 7.3);

A is the actual cross-sectional plan foundation area.

For undrained cases it can be appropriate to use a soil friction coefficient, α , to reduce the undrained soil strength at the foundation interface. The value of α varies between 0 (frictionless) and 1 (fully rough) and can be determined by specialist testing, taking due account of the roughness of the underside of the foundation.

In cases where an undrained response is anticipated in the bulk of the soil mass, it might nevertheless be appropriate to account for a drained interface between the foundation base plate and soil. Similarly, the possibility of drained or partially drained sliding along a sand seam within a competent clay layer should be considered.

The sliding resistance of a surface foundation installed on clay is heavily dependent on the undrained shear strength at the seafloor and on the contact area, both of which can be highly uncertain. Therefore, should sliding be identified as the governing failure mode, foundation skirts should be considered.

7.4.2.3 Drained conditions

Formula (12) can be used for determining drained sliding capacity at the base of the foundation (skirt tip level for skirted foundations with appropriate skirt depth to spacing ratio):

$$H_d = Q \left(\frac{\tan \phi'}{\gamma_m} \right) \quad (12)$$

where

H_d is the design resistance of sliding;

Q is the factored vertical action during the relevant loading conditions. Note that action factors of less than 1 are recommended for cases where increased vertical action has a beneficial effect on the calculated capacity;

ϕ' is the characteristic value of the effective angle of internal friction;

γ_m is the material factor (see 7.3).

Formula (12) assumes that full soil shear resistance can be mobilized along the interface between the foundation and the soil (i.e. full soil-soil contact is assumed). This should be assessed on a case-by-case basis and in some instances it can be more appropriate to consider use of an interface friction angle (δ)

between the foundation soil and the structure rather than the friction angle of the soil (ϕ'). The value of δ can be determined by specialized testing, taking due account of the roughness of the underside of the foundation compared to the seabed.

7.4.3 Assessment of torsional capacity

Torsional actions decrease the overall bearing and sliding capacity of shallow foundations. Correction factors that account for torsional actions are not available for use with the bearing capacity methods in [7.4.1](#), or the assessment of pure sliding in [7.4.2](#), and specialist advice shall be sought where this requires consideration.

Effects of torsion on foundation stability can be considered through a yield surface approach, see [A.7](#).

In assessing torsional capacity, due consideration should be given during site investigation and interpretation to the possible occurrence of discrete layers of low strength along which displacements could preferentially occur. Failure due to the formation of internal mechanisms within the confined soil plug (above skirt tip level) should also be considered.

7.5 Serviceability (displacements and rotations)

7.5.1 General

Displacements over the life of the structure shall be determined and taken into account in determining the required clearance between water level and topsides, undertaking design of connections between subsea structures, and other serviceability limits. Displacement can also affect the structural integrity of the structure.

7.5.2 Displacement under static loading

7.5.2.1 General

Calculation of foundation displacements should include:

- immediate displacements;
- primary consolidation settlement;
- secondary compression (creep) settlement;
- differential settlements induced by spatial soil variability, moments, torque and eccentricity.

The formulae for evaluating the static short-term and long-term displacements of shallow foundations are given in [7.5.2.2](#) and [7.5.2.3](#). These formulae are applicable to idealized conditions, and a discussion of the limitations is given in [A.7](#).

7.5.2.2 Immediate displacements

For soil profiles that can be assumed to be isotropic and homogeneous, for the condition where the structure base is circular, rigid, and rests on the soil surface and where the anticipated displacements are elastic, the displacements of the base of the foundation under various actions can be estimated as follows:

Vertical

$$u_Q = Q \left(\frac{1 - \nu}{4GR} \right) \quad (13)$$

Horizontal

$$u_H = H \left(\frac{7 - 8\nu}{32(1 - \nu)GR} \right) \quad (14)$$

Overturning

$$\theta_M = M \left(\frac{3(1 - \nu)}{8GR^3} \right) \quad (15)$$

Torsion

$$\theta_T = T \left(\frac{3}{16GR^3} \right) \quad (16)$$

where

u_Q is the vertical displacement at foundation base level;

u_H is the horizontal displacement at foundation base level;

θ_M is the overturning rotation (in radians) at foundation base level;

θ_T is the torsional rotation (in radians) at foundation base level;

Q is the vertical action;

H is the horizontal action;

M is the overturning moment;

T is the torsional moment;

G is a characteristic value of the elastic shear modulus of the soil (for the appropriate action and strain level);

ν is the Poisson's ratio of the soil;

R is the radius of the base of a circular foundation.

Design values of actions and moments (V , H , M and T) with an action factor = 1 should be used.

These solutions can also be used for approximating the response of a square base of equal area.

References for formulae to predict immediate, elastic displacements that account for non-uniform soil profiles (e.g. linearly increasing soil strength), foundation embedment, foundation flexibility and non-uniform base geometries are provided in [A.7](#).

Numerical analysis methods should be considered for more complex situations.

The elastic shear modulus of the soil G is not a unique soil parameter and depends on the level of stress and strain applied to each soil element. Care should be taken to select an appropriate value in design using [Formulae \(13\)](#) to [\(16\)](#). Any assumptions made in this regard shall be clearly documented. Poisson's ratio ν values are normally taken as 0,5 for undrained soil response and in the range 0,2 to 0,3 for a drained soil response.

7.5.2.3 Primary consolidation settlement

An estimate of the vertical time-dependent consolidation settlement of a fine grained layer under an imposed vertical stress can be determined by the following formula:

$$u_Q = \left(\frac{hC}{1 + e_0} \right) \log_{10} \frac{\sigma'_{v0,z} + \Delta\sigma'_{v,z}}{\sigma'_{v0,z}} \quad (17)$$

where

- u_Q is the vertical displacement at foundation base level;
- h is the layer thickness;
- e_0 is the initial void ratio of the soil;
- C is the characteristic value of compression index of the soil over the loading range considered;
- $\sigma'_{v0,z}$ is the effective overburden stress at the level of a given soil layer;
- $\Delta\sigma'_{v,z}$ is the increment of effective vertical stress in a given soil layer at the specified time.

The compression index, C_c , should be used in the calculation of consolidation settlements of normally consolidated clays. The swelling index, C_s , should be used in the calculation of consolidation settlements for highly overconsolidated clays where the relevant stress range falls on an unload-reload line. The calculation should be divided into two parts for stress ranges that span the unload-reload and normal compression lines.

Compression characteristics of the soil should be determined from appropriate consolidation tests carried out at appropriate pre-consolidation pressures. Care should be taken when selecting samples for testing, as disturbance during recovery can significantly impact the test results.

A thick homogeneous layer should be divided into multiple thin layers for analysis with each layer prescribed an appropriate value of C and e_0 . Where more than one layer is involved, the total settlement estimate is taken as the sum of the settlement of the individual layers.

[Formula \(17\)](#) is a widely used simplified estimate of long term or primary consolidation settlement obtained by assuming one-dimensional compression of soil layers under an imposed vertical stress and should be used with caution. One-dimensional consolidation theory is appropriate for prediction of consolidation settlements in soil confined by skirts and for a foundation resting on a thin layer of compressible material overlying a competent material (such as soft clay overlying sand). Specialist advice shall be sought in cases where one-dimensional assumptions are inappropriate.

The primary consolidation settlement of a foundation on clay is a three-dimensional problem in which stress distributions and pore pressures are coupled. Numerical schemes are therefore necessary where more accurate solutions than [Formula \(17\)](#) are required or where creep, loading redistributions, differential settlements or different initial conditions such as excess pore pressures need to be considered.

7.5.2.4 Secondary compression (creep)

Depending on the duration of loading, additional displacement due to secondary compression (creep) might also need to be assessed.

7.5.2.5 Differential settlements induced by eccentricity

Eccentricity of actions on a foundation can cause a permanent moment to be transferred to the foundation, leading to the potential for differential settlements, both immediately and as a result of consolidation over the life of the structure. These should be considered in the foundation design.

7.5.3 Displacement under dynamic and cyclic actions

7.5.3.1 Foundation response to applied loading

Dynamic actions involve significant inertial effects (mass-acceleration) and can be monotonic (e.g. ship impact) or cyclic (e.g. seismic). Other cyclic actions involve repetitive loading in which inertial effects are insignificant (e.g. some environmental actions and thermal actions). In some cases, cyclic actions can be considered as pseudo-static and assessed using the calculations outlined in 7.4 (e.g. current loading is often considered as pseudo-static). Other cyclic actions require more detailed analysis (e.g. to assess the mean and cyclic strain accumulation and post-cyclic volumetric strains).

7.5.3.2 Settlement after the event

In many cases, cyclic loading leads to generation of excess pore pressures at the end of the event. Dissipation of excess pore pressures leads to additional primary consolidation settlement, beyond that calculated for static loading, and can also increase the amount of creep.

7.5.4 Other contributors to foundation settlement

Additional foundation settlement, including differential settlement, can occur over the life of a structure due to, for example, subsidence of the seafloor due to oil and gas extraction or elevation changes due to on-going tectonic movements. The effect of subsidence is typically insignificant for temporary foundations.

7.6 Other design considerations

7.6.1 Hydraulic stability

7.6.1.1 Scour

Measures to prevent erosion and undercutting of the soil beneath or near the foundation base due to scour should be considered where the effects of erosion are not otherwise accounted for and where scour can detrimentally impact the performance of the foundation. Possible measures include

- accounting for/using skirts penetrating through erodible layers into scour-resistant soils or to such depths as to ensure scour does not reach the foundation base level, or
- placing scour-resistant materials around the edges of the foundation.

Sediment transport studies can be of value in planning and design.

Care shall be taken where the foundation is designed to tolerate erosion of part or all of the soil above foundation base level, because such erosion can lead to sudden and uncontrolled failure of the foundation. Due consideration shall also be given to selection of factors that contribute to foundation stability calculations, such as the assessment of passive soil resistance for skirted foundations.

7.6.1.2 Piping

The foundation should be designed to prevent the creation of excessive hydraulic gradients (piping conditions) in the soil due to environmental actions or operations performed during or subsequent to structure installation.

7.6.2 Installation, retrieval and removal

7.6.2.1 General

Installation shall be planned to ensure that the foundation can be properly seated at the intended site without unintended or excessive disturbance to the supporting soil. Penetration of foundation skirts

into the seabed can be facilitated by providing under-pressure (relative to the ambient hydrostatic pressure) inside the skirt compartments under the foundation. In the case of providing under-pressure, installation procedures shall be planned to avoid unintended disturbance to the soil, including plug uplift, erosion, and piping.

If removal is anticipated, an analysis should be made of the actions generated during removal to ensure that removal can be accomplished with the means available. Consideration should be given to increases in soil strength due to consolidation, which can enhance extraction resistance.

7.6.2.2 Skirt penetration resistance

Methods exist to predict the resistance associated with penetration of foundation skirts. Further advice is provided in [A.7](#).

8 Pile foundation design

8.1 Pile capacity for axial compression

8.1.1 General

Design criteria for pile foundations should be determined in accordance with ISO 19902.

The axial pile capacity shall satisfy the following conditions:

$$P_{d,e} \leq Q_d = Q_r / \gamma_{R,Pe} \quad (18)$$

$$P_{d,p} \leq Q_d = Q_r / \gamma_{R,Pp} \quad (19)$$

where

Q_d is the design axial pile capacity, i.e. the design resistance of the pile;

Q_r is the representative value of the axial pile capacity, as determined in [8.1](#) and [8.2](#);

$P_{d,e}$ is the design axial action on the pile [allowed to include the effective pile weight, with $(\gamma_{\text{pile}} - \gamma_{\text{water}})$ as effective unit weight, and the weight of the soil plug if this can be justified, in case of tensile loading], determined from a coupled linear structure and nonlinear foundation model using the design values of actions for extreme combinations of actions;

$P_{d,p}$ is the design axial action on the pile [including the effective pile weight, with $(\gamma_{\text{pile}} - \gamma_{\text{water}})$ as effective unit weight, in case of compressive loading], determined from a coupled linear structure and nonlinear foundation model using the design values of actions for operational combinations of actions;

$\gamma_{R,Pe}$ is the pile partial resistance factor for extreme combinations of actions ($\gamma_{R,Pe} = 1,25$);

$\gamma_{R,Pp}$ is the pile partial resistance factor for operational combinations of actions ($\gamma_{R,Pp} = 1,50$).

In accordance with ISO 19900, a material factor of 1,0 is permitted for evaluation of accidental limit states.

When sizing a pile foundation, the following items shall be considered: design actions, pile diameter, penetration, and type of tip, wall thickness, and number of piles, spacing, location, pile head fixity, material strength, installation method, and other parameters as appropriate.

The analysis procedure used shall properly simulate the nonlinear stress-strain behaviour of the soil and ensure force-displacement compatibility between the structure and the pile-soil system. Displacements and rotations of individual piles shall not exceed serviceability limit states that if exceeded would render the structure inadequate for its intended function.

Pile capacity for axial compression, as discussed in 8.1.2 to 8.1.5, relates to the axial resistance of a pile when the pile head is subjected to compressive actions along the pile axis. Pile capacity for axial tension is addressed in 8.2.

Pile capacities are commonly determined using the simplified calculation model described in 8.1.2; the parameters that are used in this model are determined in accordance with 8.1.3 to 8.1.5. For most fixed offshore structures supported on open-ended pipe piles in silica soil, experience has shown the adequacy of determining pile penetration based on static capacity evaluations, with design values of static actions and commonly accepted working stress design (WSD) factors of safety that, in part, account for the cyclic effects. The partial action and resistance factors applied for pile design in this part of ISO 19901 have been based upon these safety factors.

The simplified model for pile capacity described in 8.1.2 to 8.1.5 is based on a (quasi-)static and monotonic application of the axial actions and has no ability to reflect the complex occurrences that take place in the interaction between pile and soil during field conditions. To enhance understanding of the limitations of the model and to assist in applying engineering judgment to its results, it is useful to gain a better insight in the actual occurrences through the investigation of pile performance (see 8.3 and A.8.3).

The relationships between mobilized axial shear transfer between pile and soil and the local pile displacement, and between mobilized end bearing resistance and the pile tip displacement, can be determined using 8.4.

8.1.2 Axial pile capacity

The representative value of the axial capacity of piles in compression, including belled piles, $Q_{r,c}$, should be determined by:

$$Q_{r,c} = Q_{f,c} + Q_p = f(z) A_s + q A_{\text{pile}} \quad (20)$$

where

$Q_{r,c}$ is the representative value of the axial capacity in compression (in force units);

$Q_{f,c}$ is the representative value of the skin friction capacity in compression (in force units);

Q_p is the representative value of the end bearing capacity (in force units);

$f(z)$ is the unit skin friction (in stress units);

A_s is the side surface area of the pile in soil (m²);

q is the unit end bearing at the pile tip (in stress units);

A_{pile} is the gross end area of the pile (m²);

z is the depth below the original seafloor (m).

For open-ended pipe piles, the end bearing capacity, Q_p , shall not exceed the sum of the end bearing capacity of the internal plug and the end bearing on the pile tip wall annulus. In computing the design actions in compression on the pile, the effective weight of the pile shall be included. It is suggested to use $(\gamma_{\text{pile}} - \gamma_{\text{water}})$ as effective unit weight for a submerged pile.

In determining the capacity of a pile, consideration shall be given to the relative deformations between the soil and the pile as well as to the compressibility of the soil-pile system. In some circumstances, a more explicit consideration of axial pile performance effects on pile capacity is warranted. Additional discussion of these effects is given in 8.3 and A.8.3.

The foundation configurations should be based on those that experience has shown can be installed consistently and practically under similar conditions with the pile size and installation equipment.

being used. Possible remedial action in the event that design objectives cannot be obtained during installation should be investigated and defined prior to construction.

In the case of drilled and grouted piles, the end bearing capacity should be reduced or ignored in the design, depending on pile construction factors (such as the degree of removal of drill cuttings from the base of the hole).

In the case of belled piles, the skin friction values on the pile section should be those given in 8.1 and 8.2. Skin friction on the upper bell surface and, possibly, on the pile for some distance above the bell should be discounted in computing the skin friction resistance, $Q_{f,c}$. The end bearing area of a pilot hole, if drilled, should also be discounted in computing the total bearing area of the bell.

8.1.3 Skin friction and end bearing in cohesive soils

There are a number of methods for calculating the skin friction and end bearing in cohesive soils. The method described in this section has been developed and applied over many years and is the current industry standard. However, caution should be exercised in its application as there are many more variables which affect pile capacity than those included in the design Formulae (21) to (24). This matter is discussed in this subclause and in A.8.1.3. For driven pipe piles in cohesive soils, the unit skin friction, $f(z)$, in stress units, at depth, z , can be calculated using Formula (21):

$$f(z) = \alpha s_u(z) \quad (21)$$

where

α is the dimensionless skin friction factor, for clays

$s_u(z)$ is the characteristic value of undrained shear strength at depth z (in stress units).

The factor α can be computed by:

$$\alpha = 0,5 \Psi^{-0,5} \text{ for } \Psi \leq 1,0 \quad (22a)$$

$$\alpha = 0,5 \Psi^{-0,25} \text{ for } \Psi > 1,0 \quad (22b)$$

with the constraint that $\alpha \leq 1,0$

where

$$\Psi = \frac{s_u}{\sigma'_{vo}(z)} \text{ at depth } z \quad (23)$$

$\sigma'_{vo}(z)$ is the effective vertical stress at depth z (in stress units).

A discussion of appropriate methods for determining the undrained shear strength, s_u , and effective overburden stress, $\sigma'_{vo}(z)$, including the effects of various sampling and testing procedures is included in A.8.1.3. For underconsolidated clays (i.e. clays with excess pore pressures undergoing active consolidation), α can usually be taken as 1,0.

Due to the shortage of pile loading tests in soils having $s_u(z)/\sigma'_{vo}(z)$ ratios greater than three, Formula (22) should be applied with considerable care for high $s_u(z)/\sigma'_{vo}(z)$ values. Similar judgment should be applied for long axial flexible piles penetrating deeply in stiff soils. Low plasticity clays should be treated with particular caution (see A.8.1.3).

For long axial flexible piles some reduction in capacity might be warranted, particularly where the skin friction degrades on continued displacement. This effect is discussed in more detail in A.8.1.3.

Where the pile tip is in cohesive soils, the unit end bearing, q , in stress units, can be computed by:

$$q = 9 s_u \quad (24)$$

The skin friction, $f(z)$, acts on both the inside and the outside of the pile. The total axial resistance for pile compression is the sum of the external skin friction, the end bearing on the pile wall annulus, and the total internal skin friction or the end bearing of the plug, whichever is the lesser. For piles considered to be plugged, the bearing pressure can be assumed to act over the entire cross-section of the pile. For unplugged piles, the bearing pressure acts on the pile wall annulus only. That a pile is considered plugged or unplugged shall be based on static calculations. A pile can be driven in an unplugged condition but behave as plugged under static actions.

Skin friction resistance and end bearing capacity computed on the basis of the requirements above represent long-term capacities. Axial capacity immediately after installation is usually lower, especially in underconsolidated to slightly overconsolidated clays. This is dependent on the development of excess pore pressure in the soil during installation and its subsequent dissipation with time. When the design actions are applied to a pile foundation shortly after installation, the capacity of a pile immediately after installation and the increase in capacity with time are important design considerations. More discussion on the soil-pile set-up behaviour is provided in [A.8.1.3](#).

For piles driven in undersized drilled holes, piles jetted in place or piles drilled and grouted in place, the selection of skin friction values shall take into account the soil disturbance resulting from installation. In general, $f(z)$ shall not exceed values for driven piles; however, in some cases, for drilled and grouted piles in overconsolidated clay, $f(z)$ can exceed these values. In determining $f(z)$ for drilled and grouted piles, the strength of the soil-grout interface, including potential effects of drilling mud, shall be considered. A further check shall be made of the allowable bond stress between the pile steel and the grout, as recommended in ISO 19902.

In layered soils, skin friction values, $f(z)$, in the cohesive layers shall be as given by [Formula \(21\)](#) to [Formula \(23\)](#). End bearing values for piles tipped in cohesive layers with adjacent weaker layers can be as given in [Formula \(24\)](#) provided that

- the pile achieves penetration of two to three pile diameters or more into the layer in question, and
- the tip is approximately three pile diameters or more above the bottom of the layer to preclude punch-through.

Where these distances are not achieved, some modification of the end bearing can be necessary.

8.1.4 Skin friction and end bearing in cohesionless soils

This subclause provides a simple method for assessing pile capacity in cohesionless soils. [A.8.1.4](#) presents other recent methods which are based on direct correlations of pile unit friction and end bearing data with cone penetration test (CPT) results. In comparison to the method described in this subclause, these CPT-based methods are considered fundamentally better and have shown statistically closer predictions of pile loading test results and, although not required, are in principle the preferred methods. These CPT-based methods also cover a wider range of cohesionless soils. However, before these new methods can be recommended for routine design, more experience with them is required. CPT-based methods should be applied only by qualified engineers who are experienced in the interpretation of CPT data and understand the limitations and reliability of these methods.

For driven pipe piles in cohesionless soils, the unit skin friction, $f(z)$, in stress units, at depth, z , can be calculated by:

$$f(z) = \beta \sigma'_{vo}(z) \quad (25)$$

where

β is the dimensionless skin friction factor, for sands;

$\sigma'_{vo}(z)$ is the effective vertical stress at depth z (in stress units).

In the absence of specific data, β values for open-ended pipe piles that are driven unplugged can be taken from [Table 1](#). For full displacement piles (i.e. closed-ended or fully plugged open-ended piles) values of β can be assumed to be 25 % higher than those given in [Table 1](#). For long piles, $f(z)$ does not necessarily increase linearly with the overburden stress as implied by [Formula \(25\)](#). In such cases, it is appropriate to limit f to the values given in [Table 1](#).

For end bearing of piles in cohesionless soils, the unit end bearing, q , in stress units, can be computed using [Formula \(26\)](#):

$$q = N_q \sigma'_{vo,tip} \quad (26)$$

where

$\sigma'_{vo,tip}$ is the effective vertical stress at the pile tip (in stress units);

N_q is the dimensionless bearing capacity factor.

Recommended N_q values are presented in [Table 1](#). For long piles, q does not necessarily increase linearly with the overburden stress as implied by [Formula \(26\)](#). In such cases, it is appropriate to limit q to the values given in [Table 1](#). For plugged piles, the bearing pressure can be assumed to act over the entire cross-section of the pile. For unplugged piles, the bearing pressure acts on the pile annulus only. In this case, additional resistance is offered by friction between the soil plug and the inner pile wall. Whether a pile is considered to be plugged or unplugged shall be based on static calculations using a unit skin friction on the soil plug equal to the outer skin friction. Note that a pile can be driven in an unplugged condition, but can behave as plugged under static actions.

For soils that do not fall within the ranges of relative density and soil description given in [Table 1](#), or for materials with unusually weak grains or compressible structure, [Table 1](#) is not necessarily appropriate for selection of design parameters. For example, very loose soils or soils containing large amounts of mica or volcanic grains can require special laboratory or field tests for selection of design parameters. Of particular importance are sands containing calcium carbonate, which are found extensively in many areas of the oceans (see [A.6.4](#)).

For piles driven in undersized drilled or jetted holes in cohesionless soils, the values of $f(z)$ and q should account for the amount of soil disturbance due to installation, but they shall not exceed the values for driven piles.

In layered soils, skin friction values, $f(z)$, in cohesionless layers should be computed in accordance with [Table 1](#). End bearing values for piles tipped in cohesionless layers with adjacent soft layers can also be taken from [Table 1](#) provided that

- the pile achieves penetration of two to three pile diameters or more into the cohesionless layer, and
- the tip is at least three pile diameters above the bottom of the layer to preclude punch-through.

Where these distances are not achieved, some modification in the tabulated values can be necessary.

Table 1 — Design parameters for cohesionless siliceous soil

Relative density ^a	Soil description	Skin friction factor ^b β	Limiting unit skin friction values f_{lim} kPa (kips/ft ²)	End bearing factor N_q	Limiting unit end bearing values Q_{lim} MPa (kips/ft ²)
Very loose	Sand	Not applicable ^d	Not applicable ^d	Not applicable ^d	Not applicable ^d
Loose	Sand				
Loose	Sand-silt ^c				
Medium dense	Silt				
Dense	Silt				
Medium dense	Sand-silt ^c	0,29	67 (1,4)	12	3 (60)
Medium dense	Sand	0,37	81 (1,7)	20	5 (100)
Dense	Sand-silt ^c				
Dense	Sand	0,46	96 (2,0)	40	10 (200)
Very dense	Sand-silt ^c				
Very dense	Sand	0,56	115 (2,4)	50	12 (250)

NOTE The parameters listed in this table are intended as guidelines only. Where detailed information such as *in situ* CPT records, strength tests on high quality samples, model tests or pile driving performance is available, other values can be justified.

^a The definitions for the relative density percentage description are as follows:

Soil description	Relative density (%)
Very loose	0–15
Loose	15–35
Medium dense	35–65
Dense	65–85
Very dense	85–100

^b The skin friction factor β (equivalent to the ' $K \tan \delta$ ' term used in the past) is introduced in this part of ISO 19901 to avoid confusion with the δ parameter used in [A.8.1.4](#).

^c Sand-silt includes soils with significant fractions of both sand and silt. Strength values generally increase with increasing sand fractions and decrease with increasing silt fractions.

^d Design parameters proposed in the past for these relative density/soil description combinations can be un-conservative. Hence, CPT-based methods should be used for these soils (see [A.8.1.4](#)).

8.1.5 Skin friction and end bearing of grouted piles in rock

The unit skin friction of grouted piles in jetted or drilled holes in rock shall not exceed half the uniaxial compressive strength of the rock or grout, but in general should be much less than this value. The reduction depends on pile construction factors (such as roughness on the side of the hole) and on rock mass factors (such as the presence of discontinuities within the rock mass). The sidewall of the hole can develop a layer of slaked mud or clay, which will never gain the strength of the rock. The bond stress of the steel pile to grout interface shall be checked in accordance with ISO 19902.

The end bearing capacity of the rock shall not exceed the uniaxial compressive strength of the rock or grout multiplied by a bearing capacity factor appropriate for the type of rock. In general, the end bearing capacity should be much less or ignored in the design, depending on pile construction factors (such as the degree of removal of drill cuttings from the base of the hole), and on rock mass factors (such as the presence of discontinuities within the rock mass). The limiting end bearing capacity for this type of pile can be governed by stresses in the grout or in the pile steel.