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

This document was prepared by Technical Committee ISO/TC 67, *Materials, equipment and offshore structures for the petroleum, petrochemical and natural gas industries*, Subcommittee SC 7, *Offshore structures*.

This third edition cancels and replaces the **second** edition (ISO 19901-4:2016), which has been technically revised.

The main changes compared to the previous edition are as follows:

- guidance extended on representative and design values for soil parameters (Clause 5);
- guidance added for geotechnical design of intermediate foundations for fixed structures and clause renamed to 'Design of shallow and intermediate foundations' (Clause 7);
- requirements added on installation resistance, yield envelope approaches for ultimate limit state, and performance based design for shallow skirted and intermediate foundations (Clause 7);
- new unified CPT method for axial capacity in sands to replace the former main text method, new TZ curve definition in sands, new unified CPT method for clays introduced into the annex, new PY curve methodology for clays to replace the existing method (Clause 8);
- new clause for reassessment of pile capacity for existing structures (Clause 9);
- a new clause for pipelines, conductors and risers, previously only informative (Clause 10);
- references have been reviewed, updated and reduced where possible.

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

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

Introduction

The International Standards 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 attended and unattended 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, quality of work, 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 geotechnical design, some additional considerations apply. These include the time, frequency and rate at which actions are applied, the method of 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 structure.

These International Standards are intended to provide wide latitude in the choice of structural configurations, materials and techniques without hindering innovation. Geotechnical design practice for 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 in this document or in ISO 19900 to ISO 19906 (or elsewhere) are insufficient on their own to ensure that a safe and economical design is achieved.

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

Some background to and guidance on the use of this document is provided for information in Annex A.

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

- ‘shall’ and ‘shall not’ are used to indicate requirements strictly to be followed in order to conform 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 ‘may 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 design considerations

1 Scope

This document contains provisions for geotechnical engineering design that are applicable to a broad range of offshore structures, rather than to a particular structure type. This document outlines methods developed for the design of shallow foundations with an embedded length (L) to diameter (D) ratio $L/D < 0,5$, intermediate foundations, which typically have $0,5 < L/D < 10$ (Clause 7), and long and flexible pile foundations with $L/D > 10$ (Clauses 8 and 9).

This document also provides guidance on soil-structure interaction aspects for flowlines, risers and conductors (Clause 10) and anchors for floating facilities (Clause 11). This document contains brief guidance on site and soil characterization, and identification of hazards (Clause 6).

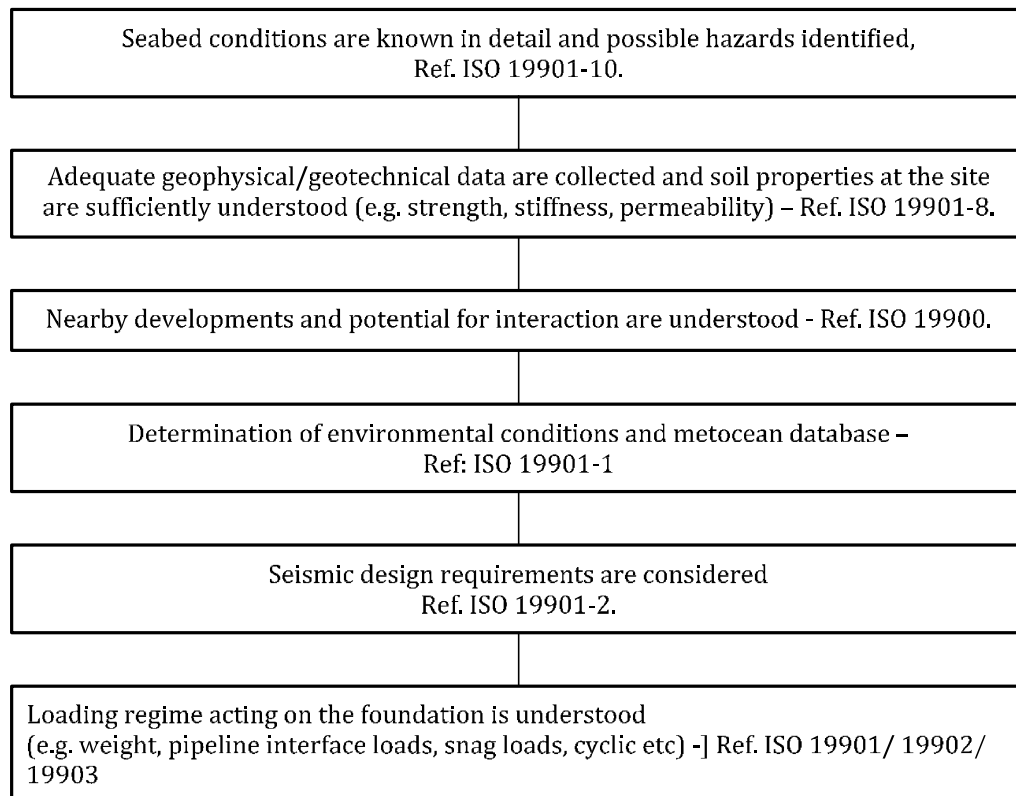
NOTE ISO 19901-8 and 19901-10 provide requirements and detailed guidance on these topics.

This document does not address aspects of soil mechanics and geotechnical engineering that apply equally to offshore and onshore structures.

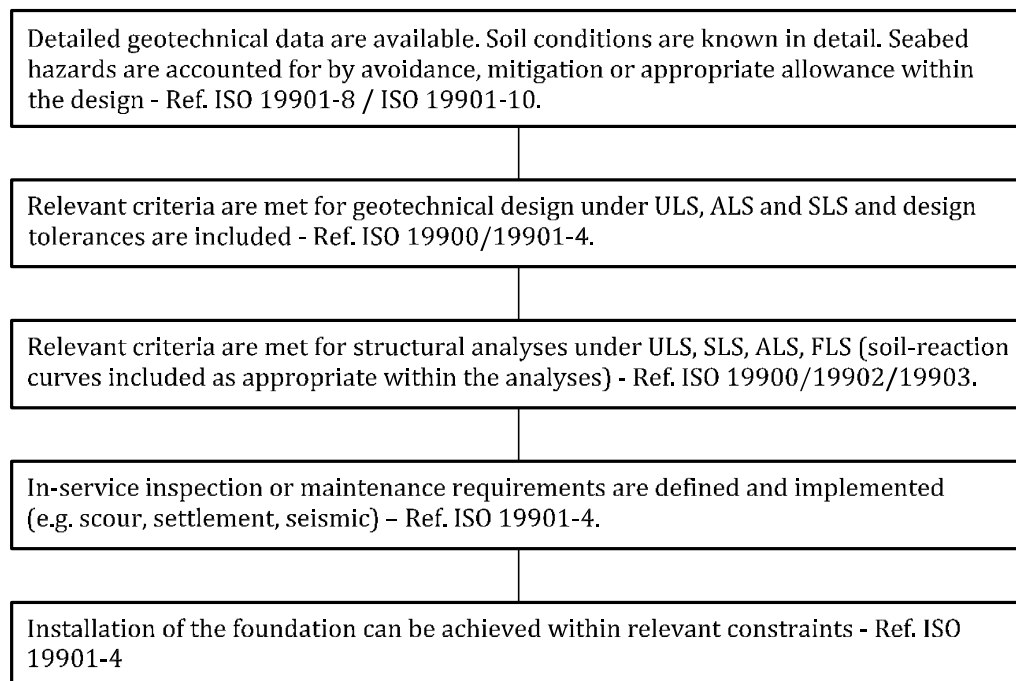
Figure 1 set outs a high level typical workflow for design of offshore foundations with reference to other relevant ISO standards.

Typical design process for offshore foundations

Collection of site condition data, foundation requirements and input data:



Foundation Design:



Iterative process from concept (initial feasibility and applicability study), basic to final design. Different level of details and objectives are required in the various design stages.

NOTE Specific design and installation constraints can apply for structures in arctic regions (see ISO 19906), for mobile offshore units, especially for jack-ups (see ISO 19905) and for anchors for floating units (see ISO 19901-7).

Figure 1 — Flowchart showing design process for offshore foundations

2 Normative references

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

ISO 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: Topsides 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 19901-10, *Petroleum and natural gas industries — Specific requirements for offshore structures — Part 10: Marine geophysical investigations*

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

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

ISO 19905 (all parts), *Petroleum and natural gas industries — Site-specific assessment of mobile offshore units*

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

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.

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

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

— IEC Electropedia: available at <http://www.electropedia.org/>

3.1

action

external loading 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 19900:2019, 3.3]

3.2

action factor

partial safety factor applied to a design action

3.3

attended

personnel are expected to work and live on the facility 24/7 during its operational life

Note 1 to entry: Usually this entails people working in day and/or night shifts and in an offshore/onshore rota of week(s) on and week(s) off the facility.

Note 2 to entry: In this document only L1 exposure level for attended structures is considered (see ISO 19900 and ISO 19902).

3.4

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 19900:2019, 3.7]

3.5

characteristic value

value assigned to a basic variable with a prescribed probability

Note 1 to entry: In some design/assessment situations, a variable can have two characteristic values, an upper value and a lower value.

[SOURCE: ISO 19900:2019, 3.9]

3.6

derived value of soil parameter

value assigned to a soil parameter for a specific calculation model, reached by reasoning, theory, deduction, inference or empiricism, often considering various sources of data

3.7

design actions

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

3.8

design value

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

[SOURCE: ISO 19900:2019, 3.14]

3.9

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]

3.10

effective foundation area

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

3.11

limit state

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

[SOURCE: ISO 19900:2019, 3.31]

3.12

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.

3.13

representative value

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

Note 1 to entry: Two types of representative value used in design verification are characteristic value and nominal value.

[SOURCE: ISO 19900:2019, 3.40]

3.14

resistance

ability of a structure, or a structural component, to withstand action effects

[SOURCE: ISO 19900:2019, 3.41]

3.15

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

3.16

scour

removal of seabed material caused by currents and/or waves

[SOURCE: ISO 19900:2019, 3.45]

3.17

seabed

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

Note 1 to entry: Offshore foundations are most commonly installed in soils, and the terminology in this document 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.

3.18

seafloor

interface between the sea and the seabed

3.19

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]

3.20

settlement

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

3.21

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.

3.22

unattended/normally unattended

no personnel are expected to work and live on the facility 24/7 during its operational life.

Note 1 to entry: As planned and/or unplanned maintenance will be required, usually reference is made to normally unattended or not normally attended.

Note 2 to entry: Specific requirements can apply to unattended/normally unattended structures or to structures being evacuated in advance of a storm.

3.23

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. Symbols can have different meanings between formulae.

4.2 Symbols for shallow and intermediate foundation 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_D	dead action factor
γ_L	live action factor
γ_m	material factor
κ	rate of increase of undrained shear strength with depth
σ'_{v0}	<i>in situ</i> effective overburden stress at foundation base level (skirt tip level when skirts are used)
$\sigma'_{v0,z}$	effective overburden stress at level of a given soil layer
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 foundation 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	mobilised lateral resistance, for lateral soil resistance-displacement relationships (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 nonlinear 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 nonlinear foundation model using the design actions for permanent and variable actions or the design axial action for operating situations
p_{fa}	mobilised lateral soil resistance-displacement relationships for fatigue analysis after the soil unload-reload secant stiffness and hysteretic damping have stabilized (p_{fa} - y_{fa} curves)
P_o	pile outer perimeter = πD
p_{mo}	mobilised lateral soil resistance-displacement relationships under monotonic lateral loading (p_{mo} - y_{mo} curves)
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
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	mobilised 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	mobilised 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
$\gamma_{R,Pe}$	partial resistance factor for extreme conditions
$\gamma_{R,Pp}$	partial resistance factor for permanent and variable actions for operating situations
Ψ	parameter to determine the dimensionless skin friction factor, for clays = $s_u(z) / \sigma'_{vo}(z)$ at depth z
$\sigma'_{ho}(z)$	<i>in situ</i> effective horizontal stress at depth z
$\sigma'_{vo}(z)$	<i>in situ</i> effective vertical stress at depth z
$\sigma'_{vo,tip}$	<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
$Q_{s\max}$	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 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
$\theta_{D'}$	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

ALS	accidental limit state
BOP	blow-out preventer
CPT	cone penetration test
FEM	finite element method
FLS	fatigue limit state
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 19900:2019, 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.

In some design situations, two representative values can be defined, an upper and a lower value. By using this approach in combination with specified action factors and material factors, geotechnical and foundation designs will result in an offshore structure with a level of reliability in agreement with the requirements of ISO 2394 and ISO 19900.

The material factor for soil can be expressed as the ratio of the representative value of the undrained shear strength to the shear stress mobilised for equilibrium, or as the ratio of the representative value of the tangent of the representative angle of internal friction to the tangent of the angle of internal friction mobilised for equilibrium. The material factor shall not be lower than 1,25. It may be modified by assessing:

- a) the consequences of failure;
- b) the accuracy of the applied calculation method and the model uncertainty;
- c) how the representative strength of the soil material was determined;
- d) whether the type of structure is new;
- e) whether there is no or little experience with the soil conditions encountered.

The PFD design methodology in this document involves the use of action factors and material factors, which aim to result in comparable overall foundation reliability with that achieved from the use of the working stress design (WSD) methodology in API RP2GEO [22]. For shallow foundations, this comparison is outlined in References [191] and [95], 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 document and API RP2GEO [22], the PFD or WSD methods can lead to differences in the design outcome.

The design shall account for static, cyclic, and dynamic actions without causing excessive deformations or vibrations in the structure. The design shall address the possibility of movement of the seabed and any actions resulting from such movements on the structure. The potential for disturbance to foundation soils by conductor installation or shallow well drilling shall be assessed (see 8.7.12).

The guidance herein does not necessarily apply to unconventional soils such as carbonate material (see 6.3), volcanic sands or highly sensitive clays.

5.2 Design cases and safety factors

The design cases that shall be addressed 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 (all parts) 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 and intermediate foundations designed with the PFD approach are given in 7.3.1 and 7.3.3. In assessing the stability of shallow and intermediate foundations with the PFD approach, the design value of resistance is computed by applying a material factor to the representative soil strength. This differs from the practice for design of piles, where a resistance factor is applied to the representative foundation capacity.

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

5.3 Representative and design values of soil parameters

5.3.1 Generic principles

This subclause provides generic principles and guidelines for selecting representative and design values of soil parameters, 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 3.17 and in ISO 19901-8 and ISO 19901-10.

Estimation of the representative and design values for soil parameters typically considers the following:

- the extent and quality of soil investigation and possible environmental factors;
- *a priori* knowledge, such as geological information and physically credible values;
- the measurable physical quantities that correspond to the population of soil parameters considered in the calculations;
- the appropriate factors or transformation functions, to convert the parameters obtained from laboratory and/or *in situ* tests or other methods to soil parameters;
- the assumptions in the calculation method(s);
- the uncertainties in the soil parameters.

Additional guidance on the selection of the representative value and uncertainties is given in 5.3.2 and A.5.3.

5.3.2 Determination of representative and design values of soil parameters

For soil parameters, the determination of the representative value and design value shall include all relevant available data for a site, such as:

- geological information;
- study of depositional processes and stratigraphy;
- results of *in situ* tests, including geotechnical and geophysical investigations (refer to ISO 19901-8 and ISO 19901-10);
- results of laboratory tests;
- results of physical model tests;
- reliable correlations (general or regional) for similar soils under similar geological settings and similar stress conditions;
- data from design tables;
- experience and expert opinions.

Figure 2 schematizes the steps for the selection of a representative value and the corresponding design value. The columns in the figure present the sources of data or information, the measurements made, obtaining derived values and obtaining the representative value and the design values. The uncertainties associated with the steps in the determination of the soil parameters are listed, although the list is not exhaustive. A design will usually require more than one basic variable and more than one representative value.

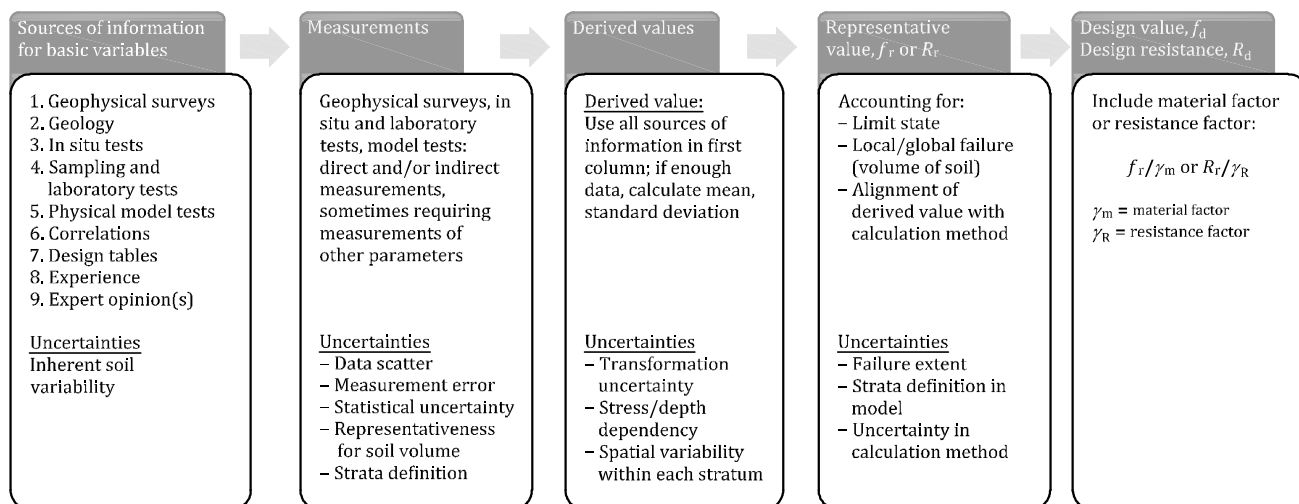


Figure 2 — Steps in selection of a representative value and design value for soil parameters

When there are enough data available, statistical analyses may be used to estimate the mean and standard deviation (uncertainty) of the derived values for key soil parameters for a design or calculation. When combining data from different sources, statistical methods, such as Bayesian updating, can be used to assess the uncertainty in the derived values of the soil parameters.

The representative value to use in the verification of a limit state in a design situation:

- is an estimate of the value(s) affecting the occurrence of the limit state which accounts for uncertainty;

NOTE 1 The uncertainty to be covered by the selection of a representative value is that of the soil parameters governing the limit state, where the uncertainty is due to both lack of knowledge and the natural and spatial variation of a parameter.

- accounts for the extent of the ground volume involved in the limit state, the effects of time, brittleness, soil fabric and structure, and construction processes;
- accounts for transformation of data to the reference scale of the soil parameter required for verification of the limit state, where applicable;
- is not one particular fractile of the results of laboratory tests or *in situ* tests on soil specimens.

NOTE 2 If statistical values in multiple parts of the volume of soil affecting the limit state are available, the representative value can be selected with a 95 % degree of confidence.

- considers both project-specific information and a wider body of geotechnical knowledge and experience;
- encapsulates the designer's expertise and experience.

As an example, when the average value of the soil strength over a large soil volume governs the design of a foundation (e.g. axial capacity of long friction piles), the representative value can be taken close to the mean shear strength value because of the larger scale variations over the volume. When foundation resistance involves a smaller volume of soil (e.g. end bearing of a pile), the representative value is usually defined below the mean value.

The following guidance is presented for selection of a representative value where estimates of the mean and standard deviation of a soil parameter are available for a uniform soil stratum [205][206][224]:

- When the representative value is based on laboratory or few discrete *in situ* data, the representative value can be selected as a value close to the mean $\pm 0,5$ SD, where SD is the standard deviation, for conditions where the limit state depends on the parameter value averaged over a large volume of ground (i.e. a mean value). For a smaller volume of ground involved in the limit state, the representative value can be selected at about the mean ± 1 SD, where SD is the standard deviation.

- When the representative value is based on nearly continuous CPT/CPTU data, and the cone resistance is used directly to compute soil resistance (e.g. for the calculation of pile capacity in sand), the representative value of the cone resistance can be taken close to the mean value, at a value of, for example, $\text{mean} \pm 0,15 \text{ SD}$. When the cone resistance is used through a transformation of the measurement into (e.g. a shear strength), the transformation introduces additional uncertainty that can be accounted for by the selection of a representative value with a larger standard deviation.

Local data, engineering experience and engineering judgment may support selection of values different than those suggested by the above guidance.

The design value is obtained by applying the material factor or the resistance factor for the calculation in question (see Figure 2). The selection of representative value does not require further adjustment of the resistance factors and material factors that are given in 7.3.1, 7.3.3 and 8.1.1.

For soils, the representative value for stability is often different than the representative value for installation (e.g. pile driving). In such cases, the representative value should be selected according to the conditions that are critical for a limit state.

For spring stiffness, lower and higher values can lead to different stresses in a structure. This implies multiple design/assessment situations, each of which require selection of appropriate representative values for soil spring stiffnesses. In many cases, a mean value, and not a cautious value, is the most appropriate as input for the structural analysis. For fatigue limit state, a value lower or higher than the mean is more appropriate.

5.3.3 Geotechnical reliability-based design

The documentation of appropriate levels of performance can be achieved for geotechnical structures and foundation by the limit state verification described in this document. Appropriate levels of performance can be alternatively documented with a reliability-based approach.

The goal of geotechnical reliability-based design (RBD) is to achieve a more uniform level of reliability than that implied in existing limit state verification approaches. Reliability-based design is particularly suitable for the representation of multiple conditions and failure modes and can account for the uncertainties found in geotechnical analysis and design. The process of using an RBD approach can give a better overall understanding of the sensitivity of the outcome to different inputs, assumptions and uncertainties. The reliability approach is well suited for comparing the safety of similar structures and for assessments during the lifetime of an installation. Methods for geotechnical RBD analyses can be found in References [21] and [37].

As a complement to the limit state verification, reliability-based approaches can be used to estimate a reliability level, described with an annual reliability index and annual failure probability, to document a margin of safety.

Reliability-based analyses should include an assessment of all the uncertainties in Figure 2. Additional required inputs for the reliability analysis are the probability distributions of the (1) soil parameter values, (2) loading cases, (3) geometry, and (4) method uncertainty. The identification of the appropriate probability distributions and their mean and standard deviation from limited data also introduces uncertainties that need to be addressed.

Additional guidance on representative values of soil properties is given in A.5.3.

5.4 Testing and instrumentation

Testing and instrumentation can be undertaken to resolve or narrow uncertainty in the behaviour of structures. Stakeholders can conclude that testing and instrumentation are required, if conditions outside industry experience are encountered and safety and economy are of particular concern.

Possible testing and instrumentation methods include the following:

- a) Loading tests, model tests and field tests

Loading tests, model tests and large-scale field tests can be performed to address particular uncertainties in the geotechnical capacity and performance, for example where:

- the structure 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 can 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 can be fitted with permanent instrumentation where:

- the safety or behaviour of the structure is dependent on active operation;
- the structure 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 structure 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 desk study, geophysical and geotechnical surveys and interpretation.

Additional guidance is provided in A.6.1, detailed guidance is provided in ISO 19901-8 and ISO 19901-10.

6.2 Geological modelling and identification of hazards

6.2.1 General

In cases where potentially active geological hazards are identified and future activity can impact proposed facilities, the nature, magnitude and return intervals of these active geological processes shall be evaluated. Site investigation techniques, specialized laboratory testing and analytical modelling can be employed to provide input for quantification of the probability and effects of active geological processes on structures and foundations. Due to uncertainties associated with definition of these processes, a parametric approach to studies can be helpful in the development of design criteria.

6.2.2 Assessment of site geohazards

Most familiar geological hazards with potential design consequences for planned facilities comprise:

- 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. more details are provided in ISO 19901-2.
- Fault planes: In some offshore areas, fault planes can extend to the seafloor with the potential for vertical and horizontal movement.

- Seafloor instability: Movements of the seafloor can be caused by ocean wave pressures, earthquakes, soil self-weight, hydrates, shallow gas, faults, salt tectonics and other geological processes.
- 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 caused by structural components disrupting the natural flow regime above the seafloor. Scour can result in removal of vertical and lateral support for foundations, causing undesirable settlements of shallow foundations and overstressing of attached components. Where scour potential is identified, it shall be accounted for in design or through mitigation [see A.8.1.4.1.2 item c) and A.8.5.6].
- Shallow gas: The presence of either biogenic or petrogenic gas in the pore water of shallow soils can be an important consideration in geotechnical design.
- 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.

Additional guidance is provided in A.6.2 and in ISO 19901-10.

6.3 Carbonate soils

6.3.1 General

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 or seabed structure behaviour can be adversely affected and a field and laboratory testing programme shall characterise these specific sediments (see A.6.3 and ISO 19901-8). Additional guidance is provided in A.6.3.1.

6.3.2 Characteristic features and properties of carbonate soils

For site characterization, use of local experience is important, particularly in the selection of an appropriate soil investigation and testing programme. In new unexplored waters, where the presence of carbonate soils is suspected, selection of an *in situ* test programme should draw upon any experience with carbonate soils where geographical and environmental conditions are similar. Additional guidance is provided in A.6.3.2.

6.3.3 Foundations in carbonate soils

6.3.3.1 Driven piles and other deep foundation alternatives

Several case histories describe some of the unusual characteristics of foundations on carbonate soils and their often poor performance. Numerous pile loading tests have shown that piles driven into weakly cemented and compressible carbonate sands and silts mobilise only a fraction of the capacity (<15 %) predicted by design and prediction methods for siliceous material.

Piles installed by driving in carbonate soils have experienced free-fall at stab-in, under hammer weight or during the driving process. The possibility of pile free-fall shall be assessed and mitigation measures, such as the use of pile arrestor or other method to reduce the speed of or arrest free-fall, shall be addressed.

Dense, strongly cemented carbonate deposits can provide a very competent foundation material, but difficulty in obtaining high-quality samples and the lack of generalized design methods can make it difficult to predict where problems can occur. Clays where the carbonate content exceeds 50 % and for which no pile test data or local experience exists can be challenging.

Additional guidance is provided in A.6.3.3.1.

6.3.3.2 Shallow foundations

Shallow foundations are suitable for use on carbonate sediments. The important differences between carbonate sediments and silica sands or non-carbonate clays shall be characterized and their effects shall be addressed in design.

Shallow foundations can be attractive for carbonate sediments that exhibit a significant degree of cementation, since they give high bearing capacities, good resistance to cyclic actions and low potential for settlements. However, layered profiles of variably cemented and un-cemented sediments can introduce the risk of a punch-through type of failure.

Additional guidance is provided in A.6.3.3.2.

6.3.3.3 Assessment

To date, general design procedures for foundations in carbonate soils are not available. Acceptable design methods have evolved but remain site-specific and dependent on local experience. Additional guidance is provided in A.6.3.3.3.

7 Design of shallow and intermediate foundations

7.1 General

Shallow foundations in the context of this document 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 up to 0,5. Intermediate foundations in the context of this document behave essentially rigidly when laterally loaded such that plastic hinges typically do not form in the steel structure of the foundation; as can be the case for longer more flexible piles that are addressed in Clause 8. Intermediate foundations are typically embedded into the seabed with an embedment between 0,5 and 10,0 times the shortest plan dimension, i.e. embedment ratios between 0,5 and 10,0.

The limits on embedment ratios stated here are indicative and there is some overlap in the foundation behaviour and design methodologies that can be applicable to shallow foundations, intermediate foundations and piles. More important is the actual behaviour of the foundation, which should be assessed by considering aspects such as the foundation dimensions, soil conditions, installation method and applied loading. A shallow foundation is typically designed considering 'surface' failure mechanisms, i.e. failure mechanisms that reach the seafloor while an intermediate foundation can be designed against mechanisms that involve both surface and confined (below the seafloor) aspects. Idealized failure mechanisms and action interaction for shallow foundations, intermediate foundations and pile foundations are illustrated in Figure 3. The simplified shallow and intermediate design methodology presented here typically assumes rigid foundation behaviour. Where there is uncertainty over the behaviour of the foundation, and the classification of foundation behaviour in terms of shallow, intermediate or pile, advanced analyses techniques, such as finite element analyses, should be used to ensure that the foundation behaviour is understood.

Guidance in this clause relates to foundations for fixed structures, whether subsea or surface piercing. Examples include shallow and intermediate foundations for surface piercing concrete gravity based structures, steel jacket structures, minimum structures, monotowers; and shallow and intermediate foundations for subsea structures, including manifolds, overtrawl structures, inline structures, pipeline end termination structures and holdback anchors for subsea structures. Foundations are considered in Clause 10 for riser towers and in Clause 11 for floating facilities. A range of structures for which the shallow and intermediate foundations considered in this clause can apply are illustrated in Figure 4. Foundations in this clause can be temporary (e.g. a construction aid during piling) or the main permanent foundation.

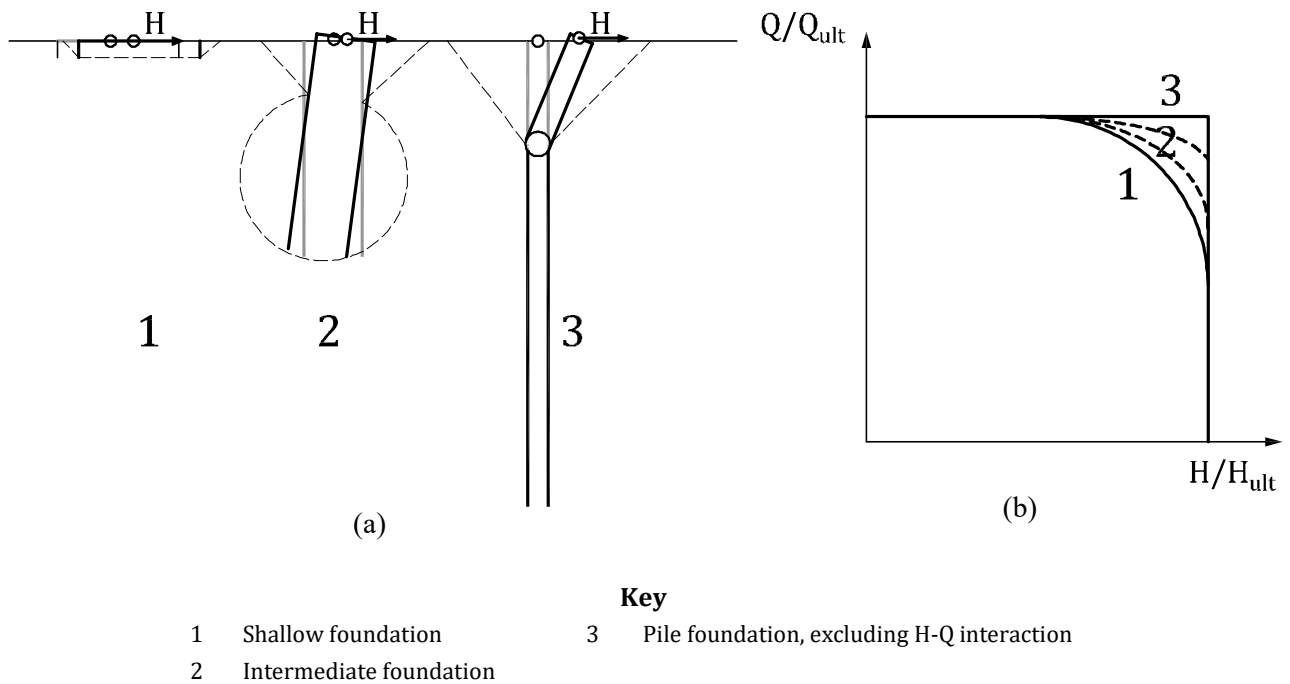


Figure 3 — Illustration of (a) failure mechanisms and (b) loading interaction of shallow foundations, intermediate foundations and flexible piles

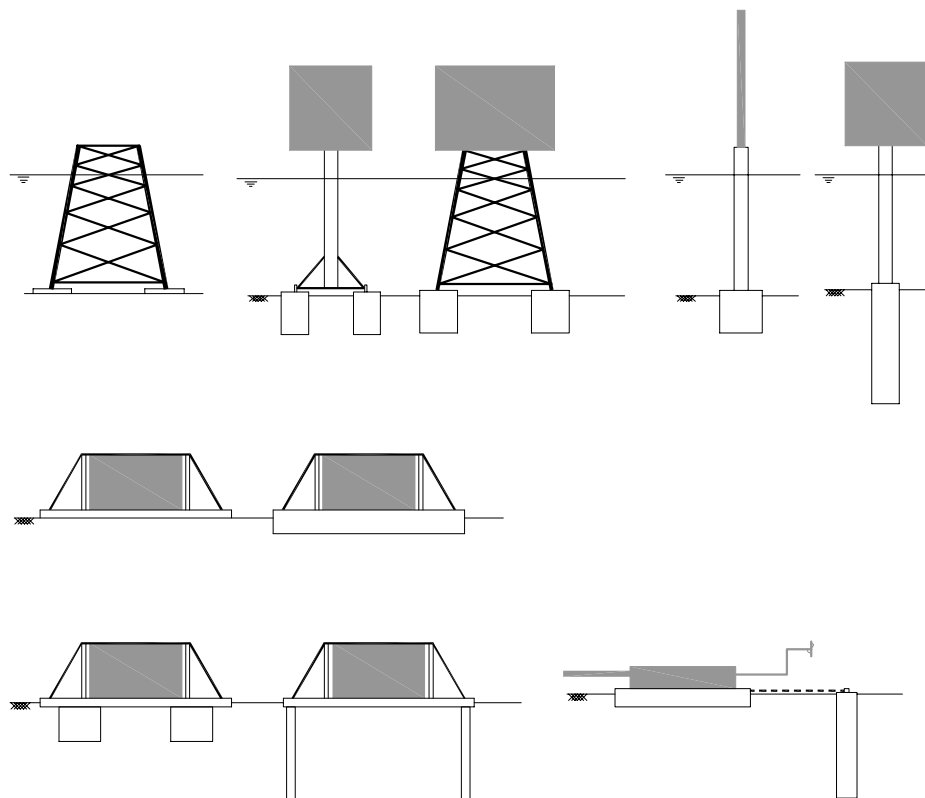


Figure 4 — Illustration of foundation types and applications considered in Clause 7

The formulae presented for evaluating the installation resistance, ultimate limit state (stability) and serviceability limit state (displacement) of shallow and intermediate foundations given 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 shall only be applied

to conditions similar to those for which they were derived or for which they can be shown to be applicable.

The methods presented are intended for combinations of dead action, variable live action and environmental action (i.e. wind, wave, current and ice). Where these arise from transient or cyclic actions, they are considered as quasi-static actions. Dynamic actions involve 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). However, the methods in this clause do not necessarily apply to more complex dynamic loading conditions, such as where the inertia of the structure or foundation soils is important (e.g. seismic loading).

This clause considers verification of limit states governed by soil behaviour and excludes verification of structural integrity of the foundation (steel jacket integrity is covered by ISO 19902). More detailed assessment can be performed for structures sensitive to foundation stiffness.

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

7.2 Principles

7.2.1 General principles

The following general principles shall be adopted in assessing shallow and intermediate foundations:

- a) Foundation stability shall be analysed by ensuring equilibrium between design actions and design resistance. Where the possibilities of excessive displacement and deformation of the foundation soil are identified, and where these are critical, more complex analysis approaches than presented in this clause can be appropriate.
- b) Calculations using alternative methods of analysis shall be justified.
- c) Design actions shall be determined with consideration of the design life of the foundation;
- d) 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 not addressed in this clause.
- e) design may 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. The selection of appropriate soil moduli (especially considering strain dependency) is essential in calculation of serviceability limit states.

7.2.2 Foundation embedment

Embedment of shallow and intermediate foundations is typically provided by peripheral vertical 'skirts' that penetrate the seabed beneath the foundation top plate. The confined soil within the skirts is typically called the 'soil plug'. 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. The term 'skirts' is used for both shallow and intermediate foundations.

Internal skirts may be provided to increase stiffness of the foundation plate and provide against a soil failure mechanism developing within the soil plug. Stability assessment based on loading at skirt tip level shall verify that an internal failure mechanisms does not form within the skirted compartment, which can lower the overall capacity. Minimum skirt spacing is covered in 7.4.2. Stiffeners may be provided along or around the skirt to increase stiffness of the embedded portion of the foundation and provide additional resistance against buckling.

The level of skirt penetration after installation shall be verified to be equal to the value assumed in design and foundation performance shall be reassessed if it is different.

Design methods presented assume full skirt penetration. The extent of skirt penetration shall be assessed. Installation resistance is covered in 7.87.8 and shall be assessed. Additional geotechnical/structural assessment is required to verify foundation performance in the event full skirt penetration is unachievable and no measure is taken to ensure full penetration.

Examples of embedment configuration for skirted shallow and intermediate foundations are illustrated in Figure 5.

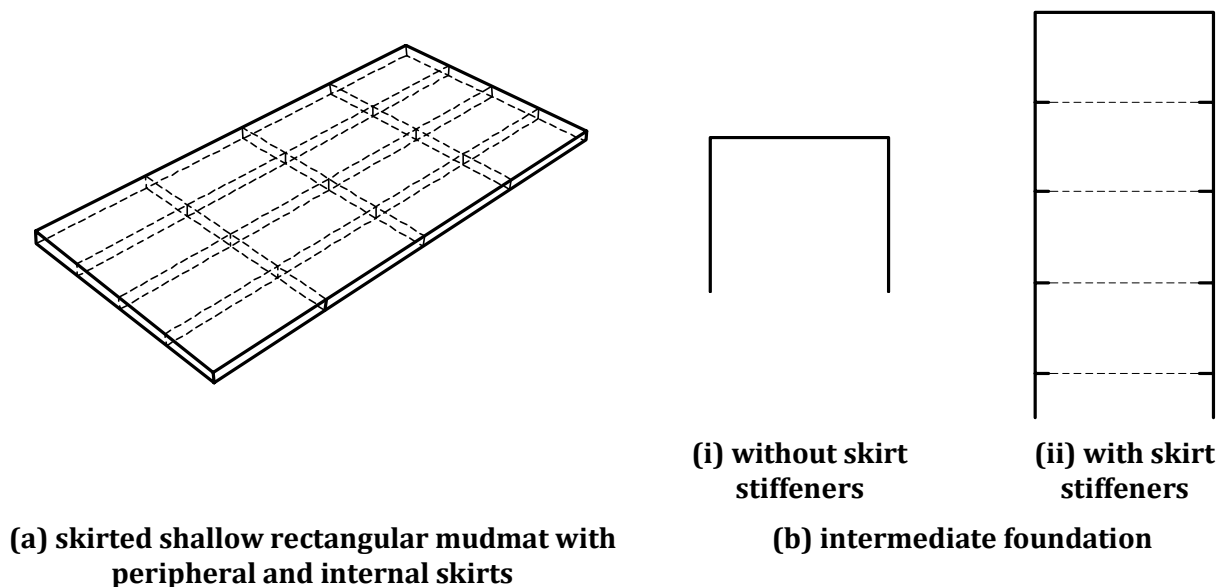


Figure 5 — Example of embedment configuration

Embedment of shallow foundations has historically been idealised by considering a surface foundation at foundation level (i.e. the level of the base of the foundation which is taken as the level of the tip of the skirts) as shown in Figure 6. Soil material properties relevant to foundation level are adopted, overburden stress is considered, and factors are applied to account for additional resistance due to the embedment. This is the case for the formulae presented in 7.5. Seafloor actions shall be transferred to skirt tip level, as described in A.7.2.3.

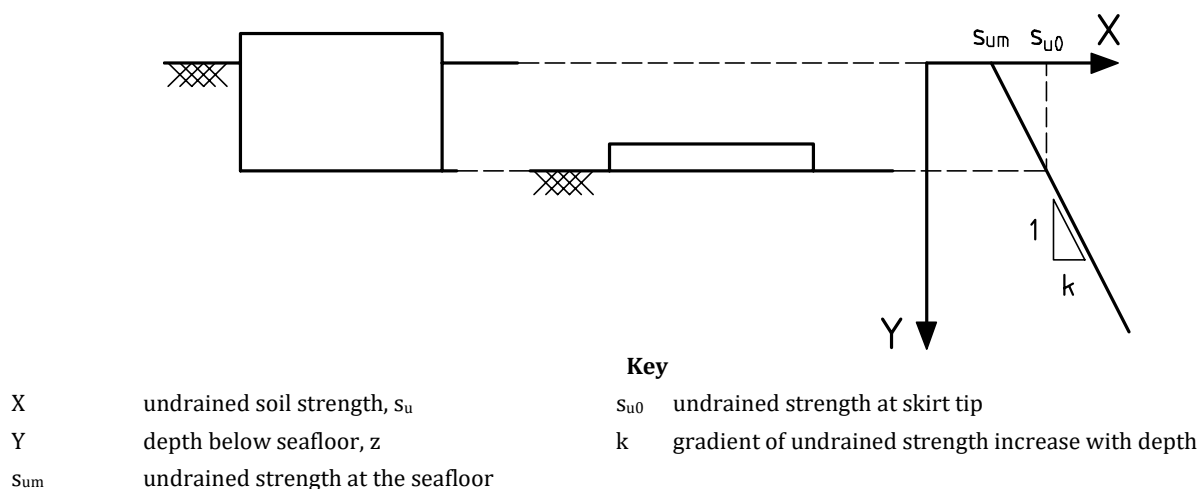


Figure 6 — Idealization of embedded shallow foundation for conventional design approach

7.2.3 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 as illustrated in Figure . This is the point of structural action transfer. H and M can be co-planar.

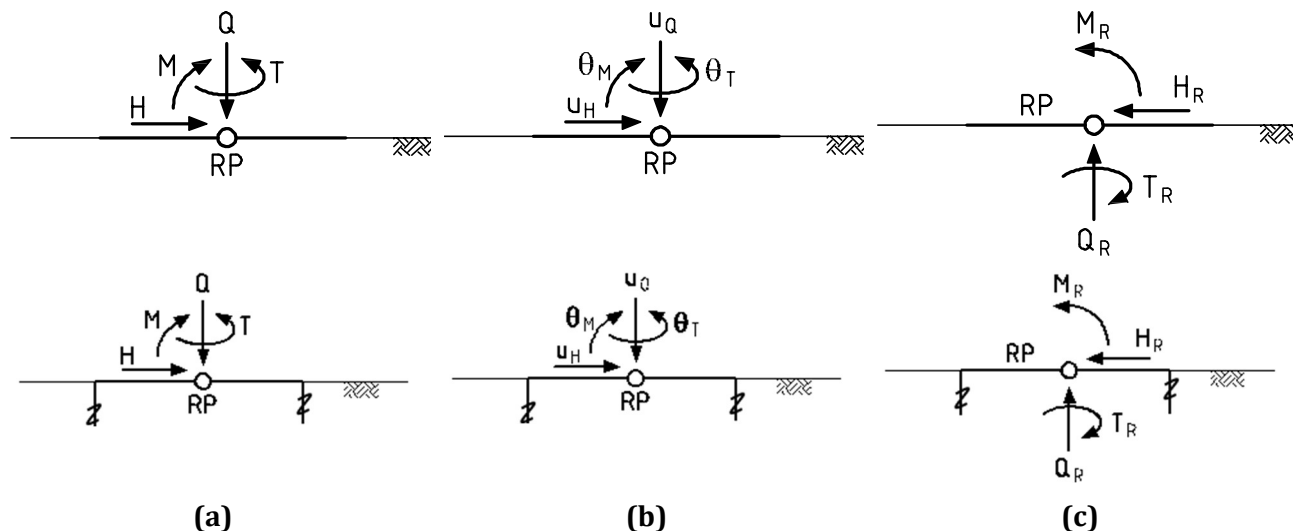


Figure 7 — Sign conventions, nomenclature and reference point for analysis of shallow and intermediate foundation (a) actions, (b) displacements and rotations, (c) seabed resistance

7.3 Acceptance criteria

7.3.1 Material and action factors

When assessing accidental limit state, the design soil strength may be determined using a material factor $\gamma_m = 1,0$ in accordance with ISO 19900.

When assessing ultimate limit state (stability), the following provisions apply:

- The design soil strength shall be determined using a minimum material factor $\gamma_m = 1,25$ except as indicated in Clause A.7. Increasing the material factor can be warranted where geotechnical data are sparse or site conditions are uncertain, or where uncertainty exists in relation to potential failure mechanisms or methods of analysis.
- Partial action factors, γ_i , shall be determined based on guidance from the relevant standard of the ISO 19900 series. The weight of the soil, including the soil plug within skirts, should normally be calculated with factors equal to unity (see 7.4.1) if it contributes to the total action. In some situations, an action factor below or above 1,0 may be justified;
- Action factors for subsea and flowline related structures shall follow the same requirements as those of fixed steel offshore structures, unless specified otherwise. Performance based design approaches may be used to justify specification of lower action factors. Further advice is provided in 7.7.2.
- Action factors and material factors shall be applied with consistency throughout the design process, respecting the physics of the loading scenario.

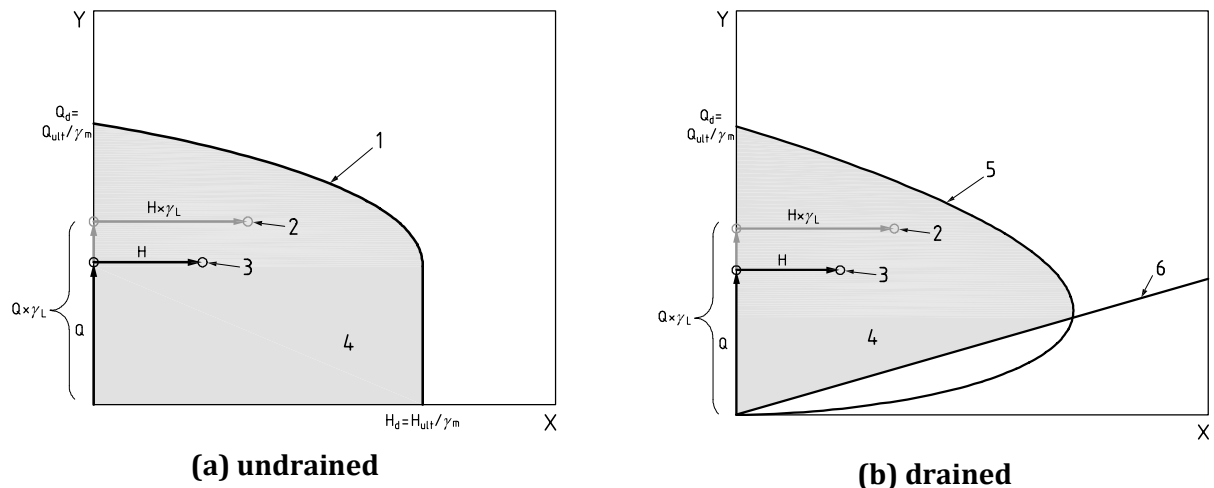
When assessing serviceability limit state (displacements), the following provisions apply:

- All action and material factors may be set to unity when calculating displacements as part of a serviceability assessment. However, where foundation displacement can lead to unacceptable consequences or governs design, factors other than unity may be used.
- Imposed deformations from external connected objects can impose or induce high actions on the foundation, or vice versa, deformation of the foundation can induce high actions because of the infrastructure it is connected to. It can be acceptable to select reduced partial action factors to

characterize the imposed loading, if the incremental deformations are small and have been robustly characterized. Further guidance is provided in 7.7.2.

7.3.2 Use of partial factors in design

A soil resistance envelope, incorporating material factors can be developed with the formulae presented in 7.5 or with numerical methods. Examples of failure envelopes for undrained and drained conditions are presented in Figure 8. Once an envelope is derived, the design actions incorporating partial action factors can be transposed onto this envelope to verify conformance with this document.



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 | allowable design actions | Y | vertical action |

Figure 8 — Soil resistance envelopes and definition of design actions under (a) undrained and (b) drained conditions

7.3.3 Special cases

For assessment of foundation stability during set-down on the seabed, stability may be assessed based on the applied vertical actions. 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 reduces the settlement and embedment of the foundation into the seabed.

7.4 Design considerations

7.4.1 Adjusting for soil plug weight

The general formulae presented to calculate ultimate limit state (stability) of shallow and intermediate foundations assume there is no difference in the depth of soil inside and outside foundation skirts. In this condition, the soil plug weight is offset by the pressure supplied by the external soil and does not contribute to the total action. 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 where significant plug heave has occurred, or 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 significant difference exists, the design vertical action may be adjusted by:

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

where

- ΔQ is the change in design vertical action to account for differences in vertical effective stress at skirt tip level, in which 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 $\gamma' z_{in}$, 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 $\gamma' z_{out}$, where z_{out} is the depth of soil outside the skirts);
- A is the actual cross-sectional plan foundation area.

A similar method can be used for embedded shallow foundations without skirts.

7.4.2 Skirt spacing

Skirts can be used on subsea foundations to:

- develop sufficient sliding capacity (especially as seafloor strengths are often low, and difficult to measure);
- protect foundation performance against undermining due to scour.

When skirts are used to develop sufficient sliding capacity, the designer may consider two options:

- 1) Use of internal and external skirts to the extent that the sliding failure mechanism is that of skirt tip sliding;
- 2) Acceptance of failure mechanism rising above skirt tip level, if the combination of sliding resistance inside the compartment and flow around the embedded members is sufficient to satisfy overall sliding capacity requirements.

When sliding is at skirt tip level, the number of internal skirts required will depend on the soil condition (e.g. seafloor strength, strength variation with depth, effects of soil consolidation), boundary conditions (e.g. drainage state of foundation underside) and loading condition (e.g. time between foundation installation, and design loading application, time-varying characteristics of the applied operational action).

Guidance on skirt spacing is provided in A.7.4.2.

When stiffeners are adopted, their effect on penetration resistance shall be assessed.

7.4.3 Foundation base perforations

Permanent or temporary perforations, in the foundation base plate are often used to:

- assist in loading out the structure through the splash zone without dynamically overloading of the associated load-out rigging;
- assist the approach of the structure to the seabed without hydraulic/hydrodynamic instability;
- assist in preventing the seabed from being unduly impacted by scour or local bearing failure.

Perforations can also be used to increase the rate of soil consolidation and settlement, and to facilitate removal of the foundation, either to allow repositioning or for end of life decommissioning.

Specific guidance is not provided on the ratio or distribution of top plate perforations. However, effects of perforations on stability shall be addressed in cases where the total area of permanent perforations exceeds 5 % of the base plate area.

References for guidance on foundation base perforations are provided in A.7.4.3.

7.4.4 Skirtless foundations penetrating into soft soils

In soft soils (e.g. normally consolidated clays), shallow skirtless 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. Incremental settlements under additional permanent actions, if any, should be addressed. Allowable differential settlement of the structure will depend upon the type of structure and its installation, and should be 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.

7.4.5 Tensile stresses beneath foundations

Reliance on tensile stresses (relative to ambient water pressure) beneath foundations that rest on the seafloor without embedment shall 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 shallow 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 may be demonstrated as acceptable in design, while longer duration tensile stresses can be carried by skirted foundations on clays with low permeability. Reliance on tensile stresses beneath a shallow foundation shall be validated with advanced methodologies, which are not explicitly addressed in this subclause.

The design shall address the level of contact between the soil and the underside of the top plate and the value of any post installation measures (e.g. grouting) to establish contact. The design shall address whether or not the foundation top cover will remain sealed for the service life of the foundation, which can influence the mobilization of reverse end bearing.

Uplift capacity may 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 demonstrated not to jeopardize the negative excess pore pressures developed during the mobilization of reverse end bearing.

7.4.6 Omni-directional actions

A single loading condition can consist of combined vertical action (Q), lateral action (H), overturning moment (M) and torsion (T), i.e. in all six degrees of freedom, when H and M are co-planar. The design can include many loading combinations.

Under such complex loading conditions, the applicability of effective area approaches, if used, shall be addressed. Yield surface approaches (see 7.7.1) and numerical analyses may be used instead.

7.4.7 Interaction with other structures

Influence of adjacent structures, such as jack-up spudcans or conductors, shall be addressed.

7.4.8 Multiple foundations

For foundations comprising several connected foundations, redistribution of loading between individual foundations generally leads to an improved system performance and may be included. The interaction between foundations can affect foundation capacity, settlement, and rotation and any detrimental effects shall be addressed in the design. Further guidance is provided in A.7.4.8.

7.4.9 Hydraulic stability

7.4.9.1 Scour

Measures to minimize erosion and undercutting of the soil beneath or near the foundation base due to scour shall be addressed where the potential for detrimental impact of scour on the foundation

performance has been identified or the effects of erosion are not otherwise accounted for. Possible measures include:

- a) using skirts penetrating through erodible layers into scour-resistant soils or to such depths that scour does not reach the foundation base level;
- b) placing scour-resistant materials around the edges of the foundation.

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

For foundations designed to tolerate erosion of part of or all of the soil above foundation base level, the effect of such erosion on foundation performance and the passive soil resistance along the skirts shall be addressed.

7.4.9.2 Piping

The foundation shall 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.

7.4.10 Unconventional soils or soil profiles

The analysis methods outlined in this document were developed primarily for use in seabed conditions comprising uniform all drained (sand) or all undrained (clay) profiles and shall not be used for unconventional soil conditions, such as:

- a) partially drained conditions;
- b) cemented material;
- c) bedrock;
- d) complex soil profiles (e.g. layered and laterally variable);
- e) carbonate soils.

The effect of seafloor unevenness shall be addressed.

Surficial crusts of stronger soil overlying weaker soil can be encountered. In such cases, accounting for the surficial crust typically reduces the foundation size. Methods to account for a surficial crust are presented in 7.5.1.5 and A.7.5.1.5.

7.4.11 Selection of soil parameter values for design

7.4.11.1 Shear strength used in stability analysis

Foundation design 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 and ISO 19901-10 provide additional information on the requirements and quality of marine soil investigations.

Uncertainty in determining the representative value of shear strength can be significant. These uncertainties are relevant for both drained and undrained shear strengths. The uncertainties shall be addressed in the selection of representative values of parameters (see 5.3, A.5.3 and A.7.5.1.5).

7.4.11.2 Parameters used in serviceability design

Parameter selection for serviceability design should consider the displacement condition being considered. For example, if calculating an upper estimate of settlements, a consistent set of soil parameters should be obtained for the most compressible soil at the site.

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

7.5 Ultimate limit state (stability)

7.5.1 Assessment of bearing capacity of shallow foundations

7.5.1.1 Failure mechanisms

For shallow foundations, bearing failure constitutes any failure mode that can 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.

7.5.1.2 Action transfer

For an embedded shallow foundation action transfer from seafloor to base level (typically skirt tip level for a foundation equipped with skirts) shall be applied in design as described in A.7.2.3.

7.5.1.3 Idealization of foundation area and the effective area concept

The formulae presented in 7.5 for shallow foundations are based on the effective area concept, defined in A.7.5.1.3, which also deals with idealization of the foundation area for use with limit equilibrium methods.

The effective area method is not intended for use with highly compressible or layered soils, or for foundations subject to high overturning moments.

With this method, actions are assumed as acting on the effective foundation area only.

7.5.1.4 Undrained conditions with constant shear strength with depth

In the absence of more definitive criteria, Formula (2) shall be used for determining the design unit bearing capacity for undrained conditions:

$$q_c = N_c \frac{s_u}{\gamma_m} K_c \quad (2)$$

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 representative 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 (2) applies to situations with approximately constant undrained shear strength to a depth equal to at least 2/3 of the foundation width.

For a vertical centric action applied to a rough-based foundation at seafloor level where both the foundation base and seafloor are horizontal, Formula (2) is reduced as follows for the following foundation shapes for appropriately factored material shear strength:

a) Infinitely long strip foundation:

$$q_d = 5,14 \frac{s_u}{\gamma_m} \quad (3)$$

b) Circular or square foundation:

$$q_d = 6,05 \frac{s_u}{\gamma_m} \quad (4)$$

7.5.1.5 Undrained conditions with linearly increasing shear strength with depth

In the absence of more definitive criteria, Formula (5) shall be used for determining design unit bearing capacity for undrained conditions with isotropic undrained shear strength increasing approximately linearly with depth under the foundation.

$$q_d = F \left(N_c s_{u0} + \frac{\kappa B'}{4} \right) \frac{K_c}{\gamma_m} \quad (5)$$

where

q_d is the design vertical bearing resistance, in which $Q_d = q_d A$;

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 representative value of undrained shear strength of the soil at foundation baseplate level (skirt tip level for skirted foundations);

K is the rate of increase of the representative value of undrained shear strength with depth;

B' is the minimum effective lateral foundation dimension (see 7.5.1.3);

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 Clause A.7.

7.5.1.6 Undrained conditions with a surface crust overlying linearly increasing shear strength with depth

Bearing capacity of a seabed condition consisting of a surficial crust of undrained shear strength, s_{uc} , over a linearly increasing shear strength, that can be idealized as illustrated on Figure 9, can be estimated as follows.

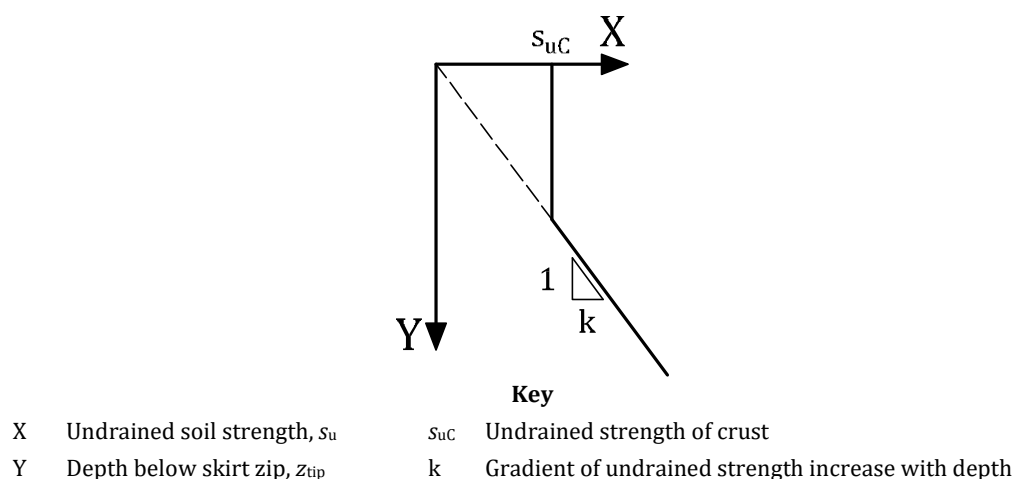


Figure 9 — Strength profile with surficial crust

The correction factor, F , in Formula (5) is modified by a correction factor to account for a surficial crust, such that:

$$F_m = F_c \times F \quad (6)$$

where

- F is the correction factor evaluated for Formula (5)
- F_m is the modified correction factor (' F_m ' replaces ' F ' in Formula (5)), accounting for the surficial crust
- F_c is the crust correction factor, detailed in A.7.5.1.6

Under complex loading conditions or more complex crust conditions, the failure envelope parameterisation presented by Reference [134] can be more appropriate.

7.5.1.7 Drained conditions

In the absence of more definitive criteria, Formula (7) shall be used for determining design vertical bearing capacity for drained conditions.

$$q_d = 0,5\gamma' B' N_\gamma K_\gamma + \sigma'_{v0} (N_q - 1) K_q \quad (7)$$

where

- q_d is the design vertical bearing resistance in the absence of horizontal actions, in which $Q_d = q_d A$;
- 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 representative value of submerged unit weight of soil;
- σ'_{v0} is the *in situ* effective overburden stress at foundation baseplate level (skirt tip level when skirts are used, taking care to correct this appropriately as per 7.4.1);
- B' is the minimum effective lateral foundation dimension (see 7.5.1.3).

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 Clause A.7.

Formula (7) 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. More advice is given in Clause A.7.

For a vertical central action applied to a foundation at seafloor level where both the foundation base and seafloor are horizontal, Formula (7) is reduced as follows for the following foundation shapes:

a) Infinitely long strip foundation:

$$q_d = 0,5\gamma' B N_\gamma \quad (8)$$

b) Circular or square foundation:

$$q_d = 0,3\gamma' B N_\gamma \quad (9)$$

7.5.2 Assessment of sliding capacity of shallow foundations

7.5.2.1 General

When assessing sliding capacity of foundations, the possible occurrence of discrete layers of low strength soil, which can provide a preferential failure surface, shall be addressed in the site characterization (investigation and interpretation).

When stability has been established using the formulae in 7.6.1, the maximum horizontal capacity shall be limited to that determined for the condition of pure sliding, as defined by Formulae (10) and (11).

If full contact between seabed and foundation is not achieved due to the configuration of the foundation or inadequate skirt penetration or any other reason, the effect on stability shall be assessed.

7.5.2.2 Undrained conditions

In the absence of more definitive criteria, Formula (10) shall 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):

$$H_d = \left(\frac{s_{u0}}{\gamma_m} \right) A \quad (10)$$

where

H_d is the design resistance for pure sliding;

s_{u0} is the representative value of undrained shear strength at foundation baseplate 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, where the failure surface occurs between the foundation and soil, a soil friction coefficient, α , ranging from 0 to 1,0 shall be applied to the undrained soil strength at the foundation interface to represent the interface friction. The interface friction can be determined by testing, taking account of the roughness of the underside of the foundation.

The possibility of drained sliding along a sand seam within a competent clay layer should be addressed.

7.5.2.3 Drained conditions

In the absence of more definitive criteria, Formula (11) shall 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):

$$H_d = Q \left(\frac{\tan \phi'}{\gamma_m} \right) \quad (11)$$

where

H_d is the design resistance of pure sliding;

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

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

γ_m is the material factor (see 7.3).

Formula (11) assumes that full soil shear resistance can be mobilized along the interface between the foundation base and the soil (i.e. a fully rough interface is assumed), and that failure does not take place within a shear band (or shear zone) in the soil. If this is not the case, it can be more appropriate to use 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 accounts for the roughness of the underside of the foundation and can be determined by laboratory testing.

If the failure takes place within the soil (i.e. within a shear band), the sliding capacity becomes also dependent on the soil dilatancy angle. If the dilatancy angle is zero, $\tan \phi'$ should be replaced by $\sin \phi'$.

7.5.2.4 Horizontal seabed resistance above foundation base level

Skirted or embedded shallow foundations can have increased resistance under pure sliding resulting from soil resistance above skirt tip level. This resistance may be used to offset horizontal actions