BS EN ISO 19901-4:2025

BSI Standards Publication

Oil and gas industries including lower carbon energy — Specific requirements for offshore structures

Part 4: Geotechnical design considerations

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This British Standard is the UK implementation of EN ISO 19901-4:2025. It is identical to ISO 19901-4:2025. It supersedes BS EN ISO 19901-4:2016, which is withdrawn.

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 committee manager.

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This document (EN ISO 19901-4:2025) has been prepared by Technical Committee ISO/TC 67 "Oil and gas industries including lower carbon energy" in collaboration with Technical Committee CEN/TC 12 “Oil and gas industries including lower carbon energy” the secretariat of which is held by NEN.

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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 document should be noted. This document was drafted in accordance with the editorial rules of the ISO/IEC Directives, Part 2 (see www.iso.org/directives).

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

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

The main changes 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 Clause A.8, new PY curve methodology for clays to replace the existing method (Clause 8);

— new requirements added on reassessment of pile capacity for existing structures (Clause 9);

— a new clause for pipelines, conductors and risers (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.

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Introduction

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

This document may be applied for the design of foundations used in the offshore wind industry. In this case, it should be verified that the type and dimension of the foundation, as well as the type of actions acting upon it, are consistent with those used in the development of the design methods. For example, the pile design methods of Clauses 8 are not necessarily applicable to the design of monopiles for which L/D is less than 10 and their validity for such cases should be assessed. Offshore wind structures can also have other requirements, such as a characterisation of foundation stiffness, that are beyond the scope of this document. Reference should be made to the overarching application specific codes and standards such as IEC 61400-3-1.

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, should be considered in relation to the overall reliability of all offshore structural systems.

For geotechnical design (engineering science dealing with the properties of soil: sand, silt, clay and rock), 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, should to be considered in relation to the overall reliability of the structure.

The International Standards on offshore structures prepared by TC 67 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 the International Standards on offshore structures prepared by TC 67 (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 in Annex A.

ISO 19905 provides requirements and detailed guidance on foundations for mobile offshore units.

Figure 1 set outs a typical workflow for design of offshore foundations with reference to other relevant International Standards.

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

Seabed conditions are known and possible hazards identified - ISO 19900 and ISO 19901-10.

Geophysical/geotechnical data are collected and soil properties at the site are sufficiently understood (e.g. strength, stiffness, durability, permeability) - ISO 19900 and ISO 19901-8.

Nearby developments and potential for interaction are assessed - ISO 19900.

Determination of physical environmental conditions (which included met-ocean.) - ISO 19900 and ISO 19901-1.

Seismic design requirements are addressed - ISO 19900 and ISO 19901-2.

Actions on the foundation are assessed (e.g. weight, pipeline interface loads, snag loads, cyclic etc) - ISO 19901/ 19902/ 19903.

Foundation Design:

Detailed geotechnical data are available - 19901-8.
Seabed hazards are accounted for by avoidance, mitigation or appropriate allowance within the design - ISO 19901-8 / ISO 19901-10.

Relevant criteria are met for geotechnical design under ULS, ALS, SLS and FLS with design tolerance included - ISO 19900/19901-4.

Relevant criteria are met for structural analyses under ULS, SLS, ALS, FLS (soil-reaction curves included as appropriate within the analyses) - ISO 19900/19902/19903.

Installation and decommissioning of the foundation can be achieved within relevant constraints - ISO 19901-4.

In-service inspection or maintenance requirements are defined and implemented (e.g. scour, settlement, seismic) - ISO 19901-4 and ISO 19901-9.

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 Design can be an 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.

Figure 1 — Flowchart showing typical design process for offshore foundations

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Oil and gas industries including lower carbon energy — 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 $0{,}5 \leq L/D \leq 10$ (see Clause 7), and long and flexible pile foundations with $L/D > 10$ (see Clauses 8 and 9).

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

This document can be applied for foundation design for offshore structures used in the lower carbon energy industry.

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 19901-7, *Oil and gas industries including lower carbon energy — Specific requirements for offshore structures — Part 7: Station-keeping systems for floating offshore structures and mobile offshore units*

ISO 19901-8, *Oil and gas industries including lower carbon energy — Offshore structures — Part 8: Marine soil investigations*

ISO 19901-9, *Oil and gas industries including lower carbon energy — Specific requirements for offshore structures — Part 9: Structural integrity management*

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

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

ISO 19904-1, *Petroleum and natural gas industries — Floating offshore structures — Part 1: Ship-shaped, semi-submersible, spar and shallow-draught cylindrical structures*

ISO 19905 (all parts), *Oil and gas industries including lower carbon energy — Site-specific assessment of mobile offshore units*

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

DNV-RP-F110, *Global buckling of submarine pipelines*

DNV-RP-F114, *Pipe-soil interaction for submarine pipelines*

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3 Terms and definitions

For the purposes of this document, the following terms and definitions apply.

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

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

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

3.1
action
external loading applied to the structure (direct action) or an imposed deformation or acceleration (indirect action)

EXAMPLE An imposed deformation can be caused by fabrication tolerances, differential *settlement* (3.18), temperature change or moisture variation. An imposed acceleration can be caused by an earthquake.

[SOURCE: ISO 19900:2019, 3.3]

3.2
action factor
partial factor whose value reflects the uncertainty or randomness of the action

3.4
basic variable
variable representing physical quantities which characterize actions and environmental influences, geometric quantities, or material properties including soil properties

[SOURCE: ISO 19900:2019, 3.7, modified — Note 1 to entry has been removed.]

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* (3.11) for use in *limit state* (3.9) verification

Note 1 to entry: Design values can be different in different design/assessment situations due to different partial factors.

Note 2 to entry: The term “characteristic value” used in ISO 19900 is not used in this document; and both terms “characteristic value” and “representative value” are considered equivalent for geotechnical and foundation design.

[SOURCE: ISO 19900:2019, 3.14, modified — Note 2 to entry has been added.]

3.7
drained condition
condition whereby the applied stresses and stress changes are supported entirely by the soil skeleton and do not cause a change in pore pressure

[SOURCE: ISO 19901-8:2023, 3.9]

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

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3.9
limit state
state beyond which the structure or structural component no longer satisfies the design/assessment criteria

[SOURCE: ISO 19900:2019, 3.31]

3.10
material factor
partial factor applied to the representative *strength* (3.19) 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* (3.4) for verification of a *limit state* (3.9) in a design/assessment situation

[SOURCE: ISO 19900:2019, 3.40, modified — Note 1 to entry has been removed.]

3.12
resistance
ability of a structure, or a structural component, to withstand action effects

[SOURCE: ISO 19900:2019, 3.41]

3.13
partial resistance factor
factor used for *limit state* (3.9) verification, the value of which reflects the uncertainty or variability of the foundation *resistance* (3.12) including those of material properties

3.14
scour
removal of *seabed* (3.15) material caused by currents, waves or ice

[SOURCE: ISO 19900:2019, 3.45, modified — "or ice" has been added.]

3.15
seabed
materials at or below the *seafloor* (3.16), 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.

3.16
seafloor
interface between the sea and the *seabed* (3.15)

3.17
serviceability
ability of a structure or structural member to perform adequately for 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.

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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 19901-8:2023, 3.44]

3.21
undrained shear strength
maximum shear stress at yielding or at a specified maximum strain in an *undrained condition* (3.20)

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

[SOURCE: ISO 19901-8:2023, 3.45]

4 Symbols and abbreviated terms

4.1 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
c'	effective cohesion
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

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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 vertical bearing capacity factor, equal to 5,14
N_q, N_γ	drained vertical bearing capacity factors, as a function of ϕ'
p'_{in}	in situ vertical effective stress at skirt tip level inside the skirts of a skirted foundation
p'_{out}	in situ vertical effective 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_d	design axial pile capacity, i.e. the design resistance of the pile
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
s_u	undrained shear strength
s_{uCR}	undrained strength of crust

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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 soil unit weight
γ_D	permanent action factor
γ_L	live action factor
γ_m	material factor
κ	rate of increase of undrained shear strength with depth
σ'_{v0}	in situ vertical effective overburden stress at foundation base level (skirt tip level when skirts are used)
$\sigma'_{v0,z}$	effective vertical stress at level of a given soil layer
ν	Poisson's ratio of the soil
υ	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

4.2 Symbols for pile foundation design

A_{pile}	gross end area of pile, $A_{pile} = \frac{\pi \cdot D^2}{4}$
A_r	pile displacement ratio, $A_r = \frac{A_w}{A_{pile}} = 1 - \left(\frac{D_i}{D}\right)^2$
A_w	cross-sectional area of pile annulus, $A_w = \frac{\pi}{4} \cdot \left(D^2 - D_i^2\right)$
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, $D_i = D - 2\ WT$

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D_{50}	mean soil particle diameter
D_{CPT}	diameter of CPT tool, D_{CPT} = 36 mm for a standard cone penetrometer with a cone area of 1 000 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 = $L - z$
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	Stress exponent to calculate Q_{tn}
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 = π D
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
$p'_m(z)$	in situ effective mean stress at depth z

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q	unit end bearing at pile tip
$q_c(z)$	representative 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 1,5D above pile tip and 1,5D below pile tip
$q_{c,tip}$	CPT cone resistance at pile tip
q_t	corrected cone resistance as defined in ISO 19901-8:2023, 8.3
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
Q_{tn}	normalized cone resistance, $Q_{tn} = \left[\left(q_t - \sigma_{v0}\right)/p_a\right](p_a - \sigma'_{v0})^n$
s_u	representative undrained shear strength
$s_u(z)$	undrained shear strength at depth z
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

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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'	representative 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 at depth *z*. $\Psi = s_u(z) / \sigma'_{vo}(z)$
$\sigma'_{\text{ho}}(z)$	in situ effective horizontal stress at depth z
$\sigma_{\text{vo}}(z)$	in situ total vertical stress at depth z
$\sigma'_{\text{vo}}(z)$	in situ effective vertical stress at depth z
$\sigma'_{\text{vo,tip}}$	in situ 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)

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

D	flowline pipeline or conductor 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, $k_v = \Delta Q / \Delta z$
N	integrated normal force
N_c	dimensionless bearing capacity factor
Q_{smax}	maximum suction (uplift) force, per unit length of pipeline

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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

4.4 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

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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

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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

4.5 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
OCR	Overconsolidation ratio
PFD	partial factor design
REB	reverse end bearing
RP	reference point for action transfer
SCR	steel catenary riser
SD	standard deviation
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
WT	pile wall thickness

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5 General requirements

5.1 General

The design methodology inherent to this standard is based on the partial factor design (PFD) determnistic approach with specified action factors and material factors (see ISO 19900). Requirements regarding partial factors for actions and the combination of actions into design situations are given in the relevant International Standards for offshore structures prepared by TC 67, i.e. ISO 19900, the ISO 19901 series, ISO 19902, ISO 19903, ISO 19904-1, the ISO 19905 series, ISO 19906.

In some design situations, two representative values can be defined, an upper and a lower value. By considering the appropriate representative value for a design in combination with specified action factors and material factors, geotechnical and foundation designs result in an offshore structure with a level of reliability in agreement with the requirements of ISO 2394 and ISO 19900. Reliability-based (5.4) and risk-informed (7.7.2) design approaches, that shall follow principles of ISO 2394 and ISO 19900 if adopted, can be used.

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 on the shear strength of the soil shall not be lower than 1,25 in deterministic PFD when assessing the ULS. It may be increased by assessing:

— the consequences of failure;

— the accuracy of the applied calculation method and the model uncertainty;

— how the representative shear strength of the soil material was determined;

— whether the type of structure is new;

— 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 (resistance fators for piles), 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 [21]. For shallow foundations, this comparison is outlined in References [182] and [85], 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 [21], 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 (soils) 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 8.7.11).

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 partial factors

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

a) ISO 19900 for offshore structures;

b) ISO 19902 for fixed steel offshore structures;

c) ISO 19903 for fixed concrete offshore structures;

d) ISO 19904-1 and ISO 19901-7 for floating offshore structures;

e) the ISO 19905 series for jack-ups;

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f) ISO 19906 for arctic offshore structures.

The resistance factors applicable to the design of pile foundations are given in 8.1.1 (also ISO 3421:2022 [176]). The material factors applicable to the design of shallow and intermediate foundations designed with the PFD approach are given in 7.3.1 and 7.3.2. 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 value of the soil shear 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 geotechnical parameters

5.3.1 Guidelines

This subclause provides guidelines for selecting the representative value of geotechnical parameters in the partial factor design (PFD) approach.

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

a) data from experience;

b) the measurable physical quantities that correspond to the types and definitions of of representative values of the geotechnical parameters required for limit state verification in the calculations;

c) the appropriate factors or transformation functions, to convert the acquired parameter values from laboratory and/or in situ tests or other methods, to geotechnical parameters;

d) the assumptions in the calculation method(s);

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 shall include site-specific data, such as:

a) geological information; depositional processes and stratigraphy;

b) the extent and quality of marine site investigation and possible environmental factors;

c) results of in situ tests, including geotechnical and geophysical investigations (refer to ISO 19901-8 and ISO 19901-10), laboratory tests, and physical model tests;

d) the uncertainties in the soil parameters required for calculations and uncertainties of parameter values available from the marine site investigations, and data from design tables;

e) method uncertainty (due to the analysis method);

f) when site-specific data are incomplete, results of correlations (general, regional or site-specific) for similar soils under similar geological settings and similar stress conditions;

g) monitoring data;

h) impact of in-service activities (e.g. consolidation, seabed disturbance).

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

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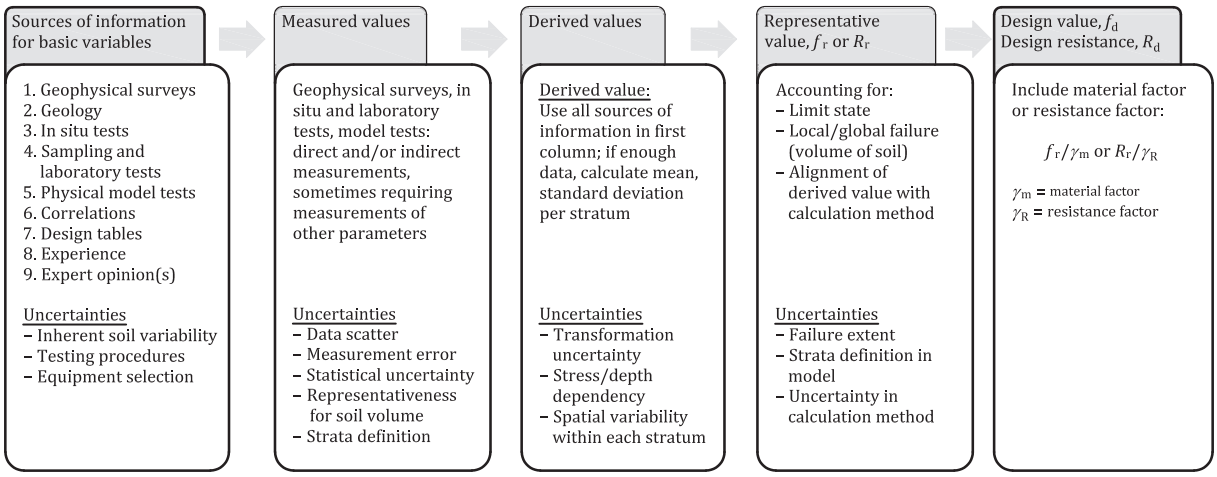

Figure 2 — Steps in selection of a representative value and design value for soil parameters

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

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 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 and soil parameter values governing the limit state, where the uncertainty is due to both lack of knowledge and the natural and spatial variation of a soil parameter, and where the soil parameter value suits the calculation method for verification of a limit state.

— accounts for the extent of the ground volume involved in the limit state, possibly 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 on soil specimens or results of in situ tests.

— considers both project-specific information and a wider body of geotechnical knowledge and experience;

— incorporates the designer's expertise and experience.

As an example of the selection of a representative value, when the average value of the soil shear strength over a large soil volume governs the design of a foundation (e.g. axial skin friction 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 derived values of a soil parameter are available for a uniform soil stratum [198][199][217]:

— 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 - 0,5 SD of the representative value, 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 of the representative value.

— 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

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value of the cone resistance can be taken close to the mean value, at a value of, for example, mean - 0,15 SD of the representative value. When the cone resistance is used through a transformation of the measurement (e.g. calculation of undrained shear strength), the transformation introduces additional uncertainty that can be accounted for by the selection of a representative value using a larger standard deviation. Local data, engineering experience and engineering judgment can support selection of values different than those suggested by the above guidance.

The selection of the representative value does not require further adjustment of the resistance factors and material factors that are given in 5.2.

For soils, the representative value for stability is often different than the representative value for installation (e.g. pile driving or skirt penetration resistance). 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 is appropriate as input for the structural analysis.

The design value is obtained by applying the material factor or the resistance factor for the calculation in question (Figure 2).

Additional guidance on representative values of soil properties is given in A.5.3.2.

5.4 Reliability-based geotechnical design

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

The goal of 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 a 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 [20] and [32].

As an additional 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:

a) soil parameter values;

b) loading cases;

c) geometry;

d) 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.

5.5 Testing and instrumentation

Testing and instrumentation can be undertaken to resolve or narrow uncertainty in the behaviour of structures. Stakeholders should consider the requirement for instrumentation or monitoring of performance during installation and operation, if conditions outside industry experience are encountered and safety and economy are of particular concern.

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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 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 requirement 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 Site investigation, identification of geohazards and carbonate soils

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.

Detailed guidance is provided in ISO 19901-8 and ISO 19901-10.

NOTE ISO 19901-8 and ISO 19901-10 provide requirements and detailed guidance on the performance of marine soil investigations and geophysical investigations for acquiring or defining the soil parameters required for design.

6.2 Geological modelling and identification of hazards

6.2.1 General

Where geohazards 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.

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6.2.2 Assessment of site geohazards

Where a geohazard is identified, it shall be accounted for in design or through mitigation. A list of potential geohazards can be found in ISO 19901-10. Examples of geohazards with potential design consequences for planned facilities include the following.

a) 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, see ISO 19901-2 which provides guidance and requirements on aspects such as design earthquakes, earthquake hazards (e.g. fault movements, liquefaction, slope instability, mud flows and rockfalls) and the design of structures and foundations in seismically active areas.

a) Fault planes: In some offshore areas, fault planes can extend to the seafloor with the potential for vertical and horizontal movement.

b) 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.

c) 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.8.1.4.2 c) and A.8.5.6].

d) Shallow gas: The presence of gas in the pore water of shallow soils can be an important consideration in geotechnical design.

e) 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.

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 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.

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Dense, strongly cemented carbonate deposits can provide a very competent foundation material, but it can be difficult to obtain high-quality samples when the degree of cementation is highly variable in a given formation (typical of carbonate deposits). In addition, 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 can be 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 for fixed structures

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, as discussed further in A.7.1.

The methods presented are intended for combinations of permanent action, variable (live) action and environmental action (i.e. wind, wave, current and ice), that can be considered pseudo-statically. 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 and ISO 19905-1. See also ISO/TR 19905-2.

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7.2 Principles

7.2.1 General principles

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

a) Foundation stability assessment shall demonstrate equilibrium between the action design value(s) and resistance design values. 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 (e.g. 7.7.1 and 7.7.2).

b) Calculations using alternative methods of analysis shall be justified.

c) Undrained calculations shall be adopted when 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 i.e. 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 when the applied stresses and stress changes are supported by the soil skeleton and do not cause a change in pore pressure i.e. 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.

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:

a) increase foundation stability;

b) decrease foundation displacement;

c) reduce impact of seabed scour on the foundation.

'Skirts' is used for both shallow and intermediate foundations.

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

Design methods presented for shallow foundations assume full skirt penetration so that the top of the internal soil plug is fully against the base of the foundation. The level of skirt penetration after installation (7.8) shall be verified to be consistent with design assumptions and foundation performance shall be reassessed if it is different.

Examples of embedment configuration for skirted shallow and intermediate foundations are illustrated in Figure 3.

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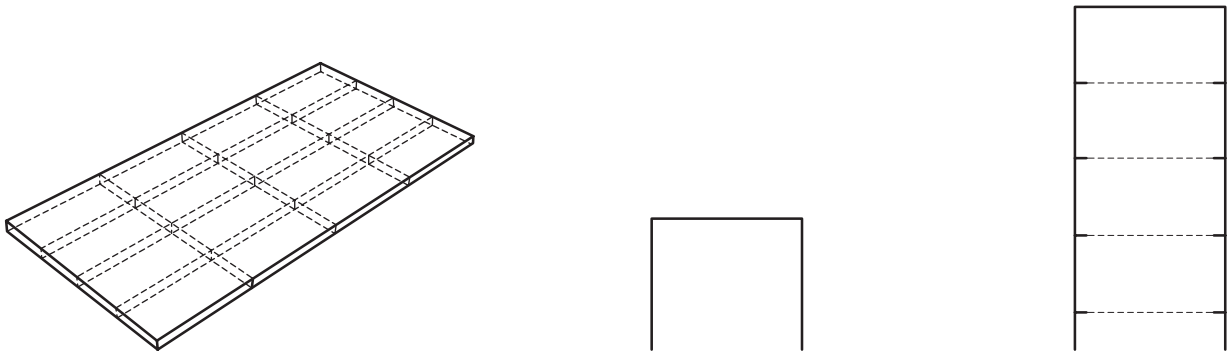
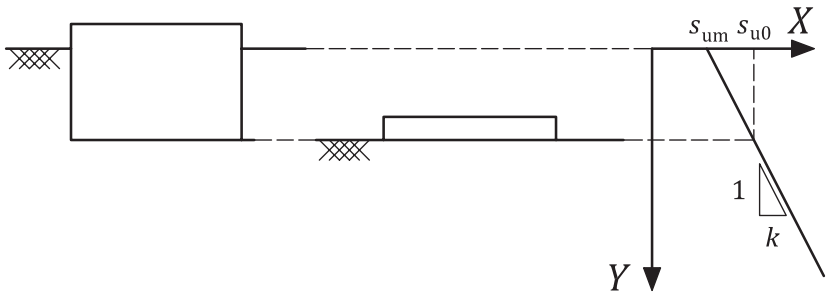


a) skirted shallow rectangular mud-mat with peripheral and internal skirts **(b) Intermediate foundation without skirt stiffeners** **(c) Intermediate foundation with skirt stiffeners**

Figure 3 — 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 4. This is the case for the methods presented in 7.5. Seafloor actions shall be transferred to skirt tip level, as described in A.7.2.3.



Key

X	undrained shear strength, s_u	s_{u0}	undrained shear strength at skirt tip
Y	depth below seafloor, z	k	gradient of undrained shear strength increase with depth
s_{um}	undrained shear strength at the seafloor		

Figure 4 — Idealization of embedded shallow foundation for conventional design approach

7.2.3 Sign conventions, nomenclature and action reference point

The load reference point and load and displacement convention shall be defined and shall be consistent with the geotechnical design method adopted.

Formulae of this clause adopt a load reference point at skirt-tip level, at the foundation geometric centroid, using a right-handed cartesian system (positively orientated vector space).

7.3 Acceptance criteria

7.3.1 Material and action factors

Partial factors shall be adopted as follows unless adopting a reliability (5.4) or risk-informed (7.7.2) design approach.

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

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

— The design value of soil strength shall be determined using a minimum material factor γ_m = 1,25. 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.

— Action factors, γ_f, shall be determined based on requirements from ISO 19901-7, ISO 19901-8, ISO 19901-9, ISO 19902, ISO 19903, ISO 19904-1, the ISO 19905 series and ISO 19906. 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.

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

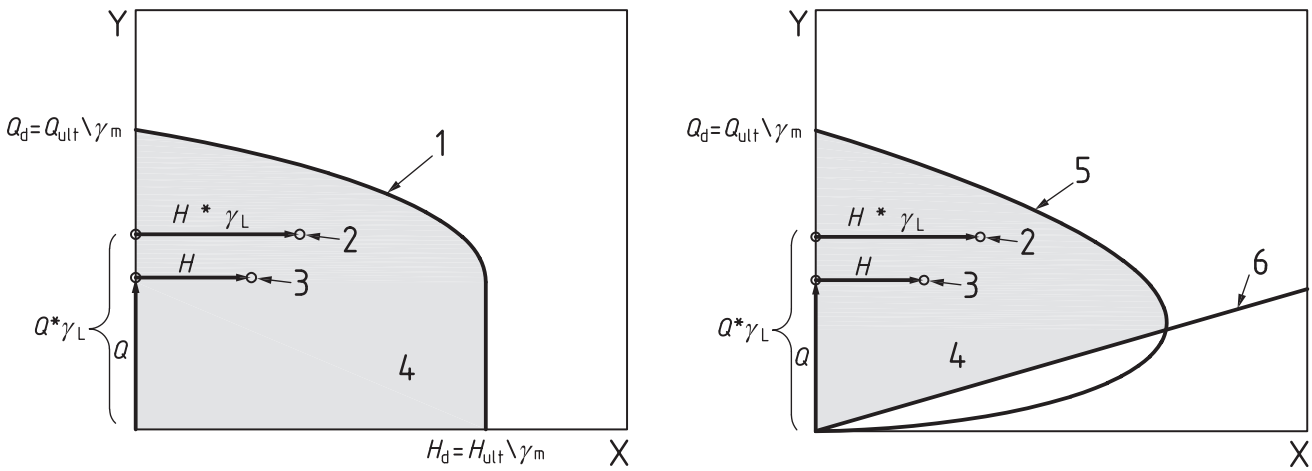
a) All action and material factors may be set to unity when calculating displacements as part of a serviceability assessment. However, where foundation displacement can lead to unacceptable consequences or governs design, factors greater than unity shall be used.

b) 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. Reduced action factors can be selected 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 method presented in 7.5 or with numerical methods. Examples of failure envelopes for undrained and drained conditions are presented in Figure 5. Once an envelope is derived, the design value of actions incorporating action factors can be transposed onto this envelope to verify conformance with this document.

Alternatively, the material factor may be applied as a resistance factor on the soil resistance envelope derived with unfactored representative soil properties.



a) Undrained conditions **b) Drained conditions**

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Key

X	horizontal action	3	applied action
Y	vertical action	4	allowable design actions
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

Figure 5 — Soil resistance envelopes and definition of design actions

A material factor of γ_m = 1,5 shall be used to assess stability at the end of foundation installation i.e. at "setdown". This assessment may be based on the vertical actions only if it is proven that:

a) horizontal and moment actions are of negligable magnitude; and

b) their neglect is of negligable conquence.

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 length of the skirts.

In cases where a meaningful difference exists, the design value of vertical action shall be adjusted by Formula (1):

$$\frac{\Delta Q}{A} = \left(p'_{\text{in}} - p'_{\text{out}}\right) \tag{1}$$

where

ΔQ is the change in design value of 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 representative value of in situ effective vertical stress at skirt tip level inside the skirts (taken as $\gamma' \times z_{in}$, where γ' is the representative value of submerged unit weight of soil and z_{in} is the representative depth of soil inside the skirts);

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

A is the representative value of the cross-sectional plan foundation area.

7.4.2 Skirt spacing

The use of skirts on subsea foundations is outlined in A.7.4.2, including guidance on skirt spacing.

The effect of the skirts and associated stiffeners on the following shall be assessed:

a) penetration resistance;

b) ultimate resistance.

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7.4.3 Foundation base perforations

Guidance on the use foundation base perforations is provided in A.7.4.3.

Effects of perforations on stability shall be addressed in cases where the total area of permanent perforations exceeds 5 % of the actual base plate area.

7.4.4 Skirtless foundations penetrating soft soils

In soft soils (e.g. normally consolidated clays), shallow skirtless foundations can penetrate the seabed to the depth at which the soil bearing resistance is in equilibrium with the applied action (e.g. buoyant self-weight), which implies no additional margin of safety. Incremental settlements under additional permanent actions, if any, shall be addressed.

7.4.5 Tensile stresses beneath foundations

Reliance on tensile stresses (relative to ambient water pressure) beneath foundations that rest on the seafloor without embedded skits 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 consider pore water pressure cavitation and 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 the foundation top cover will remain sealed for the service life of the foundation, which can influence whether transient tension can be resisted and consequently 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.

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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 at seafloor level.

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) 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 7.5.1.6 and A.7.5.1.6.

7.4.11 Selection of soil parameter values for design

7.4.11.1 Parameters used in serviceability design

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

7.4.11.2 Wave-seabed interaction

Hydrodynamic effects due the passage of large waves shall be accounted for by either appropriately adjusting the representative soil strength, or representative actions.

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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) should be applied in design.

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 compressible or layered soils, or for foundations subject to large overturning moments (relative to foundation capacity).

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 value of unit bearing capacity for undrained conditions:

$$q_{\mathrm{d}} = N_{\mathrm{c}} \frac{s_{\mathrm{u}}}{\gamma_{\mathrm{m}}} K_{\mathrm{c}} \tag{2}$$

where

q_{d} is the design value of vertical bearing resistance, and note that $Q_{\mathrm{d}} = q_{\mathrm{d}} A'$;

A' is the foundation effective area;

N_{c} is the undrained bearing capacity factor, equal to 5,14, for plane strain conditions;

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 the design value of material shear strength:

a) Infinitely long strip foundation:

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$$q_d = 5{,}14 \frac{s_u}{\gamma_m} \tag{3}$$

b) Circular or square foundation:

$$q_d = 6{,}05 \frac{s_u}{\gamma_m} \tag{4}$$

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 value of unit bearing capacity for undrained conditions with isotropic undrained shear strength increasing approximately linearly with depth under the foundation.

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

where

q_d is the design value of 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);

κ 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).

A' is the effective area.

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_{uCR}, over a linearly increasing shear strength, that can be idealized as illustrated in Figure 6, may be estimated as follows.

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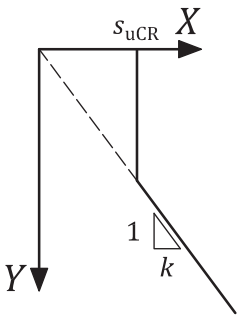

Key

X	undrained soil strength, s_u	s_{uCR}	undrained strength of crust
Y	depth below skirt zip, z_{tip}	k	gradient of undrained strength increase with depth

Figure 6 — 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:

$$F_m = F_c \times F \tag{6}$$

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 [123] 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 value of vertical bearing capacity for drained conditions.

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

where

q_d is the design value of vertical bearing resistance in the absence of horizontal actions, in which $Q_d = q_d A'$ (where A' is the effective foundation area);

N_γ, N_q are drained bearing capacity factors, as a function of the representative value 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 vertical 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).

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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.7.5.1.7.

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. Further guidance is given in A.7.5.1.7.

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:

$$q_d = 0{,}5\gamma' B N_\gamma \tag{8}$$

b) Circular or square foundation:

$$q_d = 0{,}3\gamma' B N_\gamma \tag{9}$$

7.5.2 Assessment of sliding capacity of shallow foundations

7.5.2.1 General

When discrete layers of low strength soil are identified during the site investigation and interpretation, which can provide a preferential failure surface, the sliding capacity of the foundations shall be addressed in detail.

When stability has been established using the formulae in 7.5.1, the maximum horizontal capacity shall be limited to that determined for the condition of pure sliding, as defined by Formulae (10), (11), (12) and (13).

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):

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

where

H_d is the design value of 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 representative value of the cross-sectional plan foundation area.

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 or partially drained sliding along a sand seam within a competent clay layer should be addressed.

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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):

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

where

H_d is the design value of 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;

$tan\phi'$ is the representative tangent 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 mobilised 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, the interface friction angle (δ) between the foundation soil and the structure rather than the friction angle of the soil (ϕ') shall be taken as the representative value of the effective angle of internal friction. 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 dependent on the soil dilatancy angle.

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 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.

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 reduced resistance shall be accounted for in design.

Where scour potential exists, the effect of scouring shall be addressed in design.

Formulae (12) and (13) are for the case where embedded foundations are considered as surface foundations on a reduced seafloor (see Figure 4), and are relevant for a horizontally translating foundation, i.e. without rotation. Rotation of the foundation reduces the mobilizable horizontal resistance.

General formulae for the additional horizontal resistance, ΔH_d, that can be mobilized between seafloor and foundation base level are presented in Formulae (12) and (13). Total horizontal sliding resistance is given by $H_d + \Delta H_d$.

In the absence of more definitive criteria, for undrained conditions, the additional horizontal resistance shall be calculated using Formula (12):

$$\Delta H_d = K_{ru}\left(\frac{s_{u,ave}}{\gamma_m}\right)A_h \tag{12}$$

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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.7.5.2.4);

$s_{u,ave}$ is the representative 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.

In the absence of more definitive criteria, for drained conditions, the additional horizontal resistance shall be calculated using Formula (13):

$$\Delta H_d = K_{rd}\,(0{,}5\gamma' D_b)\,A_h \tag{13}$$

where

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

K_{rd} is the drained horizontal soil reaction coefficient, which includes the material factor (see A.7.5.2.4);

γ' is the representative 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.

7.5.2.5 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 7.5.1, or the assessment of pure sliding in 7.6.2. Effects of torsion on foundation stability can be considered through a yield surface approach as described in Clause A.7 and Reference [126], or through analytical approaches as described in Reference [121]. Torsion is also considered in ISO 19906.

When discrete layers of low strength soil are identified during the site investigation and interpretation, which can provide a preferential failure surface, the torsional capacity can be critical and shall be addressed in detail. Design for torsion shall address the foundation-seabed contact stress distribution and drainage condition. Internal mechanisms within the confined soil plug (above skirt tip level) that can affect torsion capacity shall also be addressed.

7.5.3 Assessment of capacity of intermediate foundations

Use of shallow foundation methods can be suitable to provide an initial size for an intermediate foundation where loading is simple vertical or simple horizontal and the soil conditions are simple (i.e. no significant layering).

For other soil conditions or loading, including significant moments or combined loading (e.g. combined Q, H, M, T loading), more appropriate methodologies are available in Reference [192], and the sizing shall be validated by use of alternative methodology, such as finite element analyses or finite element limit analysis.

Methodologies for combined loading of intermediate foundations or piles derived for anchoring applications with a padeye located at an optimal depth below seafloor, are not generally suitable for intermediate foundations for fixed structures where the loading is at topcap level. Beam-column analyses using only p-y soil reaction curves as per the design of slender (flexible) piles under lateral loading (see 8.5) is not

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generally applicable to rigid intermediate foundations (e.g. rigid non-slender piles), unless more advanced case-specific soil reaction curves are adopted (see 8.5.6).

7.6 Serviceability limit state (displacements and rotations)

7.6.1 General

The clearance between water level and topsides, the design of connections between subsea structures, and other serviceability limits shall account for the displacements of the foundation over the life of the structure.

7.6.2 Serviceability of shallow foundations under static loading

7.6.2.1 General

Calculation of foundation displacements and rotations can include:

a) immediate displacements and rotations;

b) primary consolidation settlement (displacements and rotations);

c) secondary compression (creep) settlement;

d) differential settlements induced by spatial soil variability, moments, torque and eccentricity.

The formulae for evaluating the static short-term and long-term displacements and rotations of shallow foundations are given in 7.6.2.2 and 7.6.2.3. These formulae are applicable to idealized conditions, and a discussion of the limitations is given in Clause A.7.

7.6.2.2 Immediate displacements and rotations

Displacements of the base of a circular, rigid, foundation that rests on the surface of an isotropic and homogeneous seabed, and where the anticipated displacements are elastic, shall be estimated by Formulae (14) to (17) in the absence of more definitive criteria:

a) vertical:

$$u_Q = Q\left(\frac{1-v}{4GR}\right) \tag{14}$$

b) horizontal:

$$u_H = H\left(\frac{7-8v}{32(1-v)GR}\right) \tag{15}$$

c) overturning:

$$\theta_M = M\left(\frac{3(1-v)}{8GR^3}\right) \tag{16}$$

d) torsion:

$$\theta_T = T\left(\frac{3}{16GR^3}\right) \tag{17}$$

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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 representative 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 (Q, H, M and T) determined with a action factor of 1,0 should be used.

Formuale (14) to (17) 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 Clause A.7.

Numerical analysis methods are readily available and should be applied 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. An appropriate value in design using Formulae (14) to (17) shall be adopted and shall be documented. In the absence of more definitive criteria, Poisson's ratio ν values of 0,5 for undrained soil response and in the range 0,2 to 0,3 for a drained soil response shall be applied.

7.6.2.3 Primary consolidation settlement

Formula (18) is a widely used simplified estimate of long term or primary consolidation settlement obtained by assuming one-dimensional compression of layers of fine-grained soil under an imposed vertical stress. In the absence of more definitive criteria, Formula (18) shall be used to estimate primary consolidation:

$$u_Q = \left(\frac{hC}{1+e_0}\right)\log_{10}\frac{\sigma'_{v0,z}+\Delta\sigma'_{v,z}}{\sigma'_{v0,z}} \tag{18}$$

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 representative value of compression index of the soil over the loading range considered;

$\sigma'_{v0,z}$ is the effective vertical stress at the midpoint-level of a given soil layer;

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$\Delta\sigma'_{v,z}$ is the increment of effective vertical stress in a given soil layer at the specified time.

The representative layer thickness, h, for the calculation shall exceed the width (or diameter) of the footing. This layer thickness shall be subdivided 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.

The representative value of compression index, C, in Formula (18), shall be selected as follows. C_c, shall be used in the calculation of consolidation settlements of normally consolidated clays. The swelling index, C_s, shall be used in the calculation of consolidation settlements for fine-soil where the relevant stress range falls on an unload-reload line without reaching the normal compression line. The calculation should be divided into two parts for stress ranges that span the unload-reload and normal compression lines.

For designs sensitive to primary consolidation, compression characteristics of the soil shall be determined based on results from consolidation tests carried out at appropriate pre-consolidation pressures. However, sampling/ probe disturbance can significantly impact test results. Assumptions and corrections adopted in development of selected compression characteristics shall be documented.

Formula (18) has application limitations and does not address three-dimensional flow and strain, creep, loading redistributions, differential settlements or different initial conditions such as excess pore pressures.

7.6.2.4 Secondary compression: creep

Depending on the duration of loading and the sensitivity of the design to settlement, additional displacement due to secondary compression (creep) can be significant. In these cases, creep shall be addressed.

7.6.2.5 Differential displacements and rotations

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. Differential displacements can also derive from changing soil conditions across a foundation footprint or across individual foundations on a connected structure. Differential displacements shall be addressed in design, if the foundation or structure are sensitive to such settlements.

7.6.3 Serviceability of intermediate foundations

Where required for structural analyses or serviceability requirements, an initial estimate of foundation stiffness can be made using simplified elastic solutions. Where soil layering or loading is complex or where the situation is critical, the use of a more appropriate methodology shall be employed in detailed design. For monopile foundations, a check for accumulated displacements or rotations is be required.

7.6.4 Serviceability in response to dynamic and cyclic actions

In many cases, cyclic loading leads to accumulated excess pore pressures during 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. Settlement associated with the effects of cyclic actions shall be addressed in design where these actions can result in settlements impactful to the design solution.

7.7 Alternative methods of design

7.7.1 Yield surface approach

Offshore foundations can experience a wide range of loading, encompassing combinations of vertical action (Q), lateral action (H), overturning moment (M) and torsion (T). The conventional design approach for shallow foundations involves transforming the combined action into an equivalent vertical and lateral loading acting on a reduced (effective) foundation area. An alternative approach is to derive a fully encompassing yield surface in Q, H, M and T space, which enables foundation geometry and load interaction to be explicitly considered, as well as consideration of tensile resistance. Further guidance on use of the yield surface approach is provided in A.7.7.1.

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7.7.2 Risk-informed decision making

In a risk-informed decision making approach decisions shall be optimised with due consideration of the total risks, considering (but not limited to) the loss of lives and personal injuries, damage to the environment and monetary loss. A risk-informed design approach shall holistically consider the robustness of the system into which a foundation is connected. System failure modes shall be identified and their consequence assessed. This assessment shall consider the duration of functionality that the foundation shall provide.

Further guidance is provided in ISO 2394 and A.7.7.2.

7.8 Installation

7.8.1 General

Openings in the foundation and skirts shall be provided (if required) to accommodate drainage of entrapped water within skirt compartments, during installation (7.4.3). The openings shall not compromise the subsequent performance and satisfactory capacity of the foundation post installation. Where relevant, the risk of hydroplaning of the foundation, during lowering at/or close to touchdown, shall be assessed.

Lowering operations, near seafloor, shall be undertaken in a manner to ensure that the disturbance to seabed or founding layer is minimised and that foundation capacity and installation are not adversely affected.

The effects of seafloor topography, seafloor features and underlying strata shall be taken into account in the installation design.

Force needs to be applied to penetrate foundation skirts and any other protrusions (e.g. stiffeners) below the seafloor as the soil will be displaced to accommodate the skirts and protrusions. The foundation self-weight can provide sufficient force, but when this is not the case, penetration can also be facilitated by providing under-pressure (relative to the ambient hydrostatic pressure) inside the skirt compartments under the foundation, or temporary additional static weight (ballast).

Installation shall be planned so that the foundation can be seated at the intended site without disturbance to the supporting soil compromising the soil integrity. When providing under-pressure, installation procedures shall be planned to avoid unintended disturbance to the soil, including plug uplift, erosion and piping. Installation planning shall address risks related to installation and shall implement measures to mitigate these risks.

The foundation baseplate is not always at seafloor level after installation, as the soil displaced by the skirts can cause internal plug heave within the foundation compartments. Typically it is assumed that:

a) half of the displaced volume of external/perimeter skirts contributes to internal plug heave, if no under-pressure is used;

b) full displaced volume of external/perimeter skirts contributes to internal plug heave, if under-pressure is used;

c) full displaced volume of internal skirts contributes to internal plug heave.

7.8.2 Skirt penetration resistance

Forces acting on a foundation penetrating the seabed are illustrated in Figure 7 a). These forces are typically approximated as a bearing component (and associated overburden) on the skirt/protrusion tip, and friction component on the skirt side.

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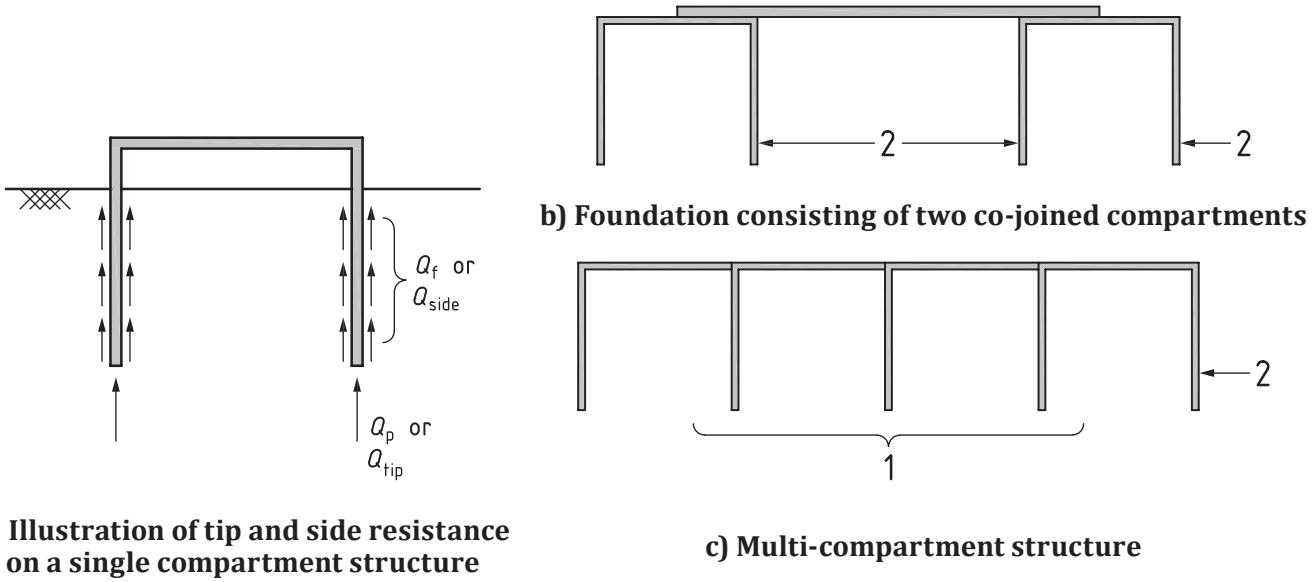

a) Illustration of tip and side resistance on a single compartment structure

b) Foundation consisting of two co-joined compartments

c) Multi-compartment structure

Key

Q_f / Q_{side}	side/shear resistance acting on skirt side	1	internal skirt(s)
Q_p / Q_{tip}	tip resistance acting on skirt tip	2	external skirt(s)

Figure 7 — Foundation penetration

Methods exist to predict the resistance associated with penetration of foundation skirts. Further guidance is provided in A.7.8.2 and A.11.5.2.2.1. Stiffeners can be required to provide satisfactory action transfer in situ, and during installation (e.g. during load-out).

7.8.3 Required and allowable under-pressure

When under-pressure is used to assist foundation installation, the following two quantities shall be assessed for all penetration depths, in addition to the penetration resistance:

a) the under-pressure necessary to allow embedment;

b) the critical pressure that can cause the soil plug to fail in reverse end bearing (clays) or piping (sands).

The pumping system used during installation shall be capable of generating the necessary under-pressure.

The following aspects shall be addressed:

— soil variability, layering and presence of any subsurface obstructions, such as cemented layers, or cobbles/ boulders;

— assurance of minimum self-weight penetration so that a seal is formed prior to under-pressure application;

— prevention of plug uplift during under-pressure assisted penetration, which can occur due to reverse end bearing (soil flow into the compartment due to excessive under-pressure);

— avoidance of piping failures through sand materials;

— avoidance of cavitation, if in shallow water.

Installation design of foundations which consist of multiple compartments or multiple co-joined foundations installed simultaneously [see Figure 7 b)] shall address risk of non-uniform penetration rate and out of verticality of penetration.

Further guidance is provided in A.7.8.2 and A.11.5.2.2.1.

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7.9 Relocation, retrieval and removal

Foundations occasionally need to be relocated if the required installation tolerances (e.g. verticality) are not achieved. The offset for the new location should be a centre-to-centre spacing of three times the foundation diameter/width. A lesser spacing may be acceptable, if the level of seabed disturbance due to the first installation is minimal.

Recovery of the foundation should be performed in a slow and controlled manner, preferably lifting initially from one corner or side of the foundation, to reduce suction effects, in addition to facilitating sufficient drainage.

If removal is anticipated, an analysis shall be made of the actions generated during removal so that removal can be accomplished with the available equipment. The analysis shall include increases in soil strength due to consolidation from the time of installation to the time of extraction.

Further guidance for over-pressure assisted removal is provided in A.11.5.2.2.2.

8 Pile foundation design

8.1 Pile capacity for axial compression

8.1.1 General

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

The axial pile capacity shall satisfy the following conditions:

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

$$P_{d,p} \leq Q_d = Q_r/\gamma_{R,Pp} \quad (20)$$

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 [which may include the effects of the effective pile weight, with ($\gamma_{pile} - \gamma_{water}$) as effective unit weight], and the weight of the soil plug if this can be justified, [in case of tensile actions], determined from a coupled structure and 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 ($\gamma_{pile} - \gamma_{water}$) as effective unit weight, in case of compressive actions], determined from a coupled structure and 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 may be used for evaluation of accidental limit states.

When sizing a pile foundation, the following items shall be addressed:

a) design actions;

b) diameter;

c) penetration

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d) type of tip;

e) wall thickness;

f) number of piles;

g) spacing;

h) location;

i) pile head fixity;

j) material strength;

k) installation method.

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.

The design methods presented in 8.1.3 and 8.1.4 only apply to clay soil profiles and sand soil profiles, respectively. They may also be used for layered profiles of sands and clays. The methods shall not be applied to other soil types or unconventional soils without confirming suitability. The method in 8.1.4 shall not be applied to sands predominately composed of weak grains and a compressible structure which exhibit degradation of strength and stiffness under cyclic loading, such as carbonate sands.

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, 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 imply non-detrimental cyclic effects. The action factors and partial resistance factors applied for pile design in this document 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.

The calculated pile capacity takes time to develop after installation. If the design actions are applied to a pile foundation before its calculated capacity has fully developed, the pile capacity shall be adjusted.

8.1.2 Axial pile capacity

In the absence of more definitive criteria, the representative value of the axial capacity of piles in compression, $Q_{r,c}$, shall be determined by:

$$Q_{r,c} = Q_{f,c} + Q_p = f(z)\, A_s + q\, A_{pile} \tag{21}$$

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 (m^2);

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

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

z is the depth below the original seafloor (in length units).

For open-ended pipe piles in clay, 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. Guidance on unplugged end bearing in sands is provided in 8.1.4. In computing the design actions in compression on the pile, the effective weight of the pile shall be included.

Formula (21) assumes that the pile behaves rigidly and can overestimate the capacity of long axial flexible piles if the skin friction degrades on continued displacement. In determining the capacity of a pile, the relative deformations between the soil and the pile as well as the compressibility of the soil-pile system shall be addressed. If the determination of the pile penetration is sensitive to the pile flexibility, the impact of pile flexibility and pile performance on the pile capacity shall be addressed. Further guidance of these effects is provided in 8.3 and A.8.3.

The foundation type and geometry 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 in rocks, the representative end bearing capacity is often reduced or ignored in the design and shall account for pile construction factors, such as the degree of removal of drill cuttings from the base of the hole.

8.1.3 Skin friction and end bearing in clay soils

There are a number of methods for calculating the skin friction and end bearing in clay soils. The method described in this section has been developed and applied over many years and is the current industry standard. However, there are more variables which affect pile capacity than those included in the design Formulae (22) to (24). This matter is discussed in this subclause and in A.8.1.3. An alternative CPT-based method that can be used for pile capacity in clays is presented in A.8.1.3.2.2.

In the absence of more definitive criteria, for driven steel open-ended pipe piles in clay soils the unit skin friction in tension and compression, $f(z)$, in stress units, at depth, z, shall be calculated using Formula (22).

$$f(z) = \alpha s_u(z) \tag{22}$$

where

α is the dimensionless skin friction factor, for clays;

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

The factor α shall be computed by:

$$\alpha = 0{,}5\ \Psi^{-0{,}5} \text{ for } \Psi \leq 1{,}0 \tag{23a}$$

$$\alpha = 0{,}5\ \Psi^{-0{,}25} \text{ for } \Psi > 1{,}0 \tag{23b}$$

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

where

$$\psi = \frac{s_u}{\sigma'_{v0}(z)} \text{ at depth } z \tag{24}$$

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

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A discussion of 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 under-consolidated clays (i.e. clays with excess pore pressures undergoing active consolidation), α may 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, the justification for application of Formula (22) shall be documented. Similar justification shall be documented for the application of Formula (22) in low plasticity clays (see A.8.1.3).

For long axial flexible piles the relative deformations between the soil and the pile as well as the compressibility of the soil-pile system shall be addressed. If the determination of the pile penetration is sensitive to the pile flexibility, the impact of pile flexibility and pile performance on the pile capacity shall be addressed. This effect is discussed in more detail in A.8.1.3.

In the absence of more definitive criteria, where the pile tip is in clay soils, the unit end bearing stress, q, shall be computed by:

$$q = 9\, s_u \tag{25}$$

The skin friction, $f(z)$, acts on both the inside and the outside of the pile. The inside friction is equal to the outside friction and is assumed not to be affected by changes in wall thickness. 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 under-consolidated to slightly over-consolidated 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 shall be addressed in design. Further guidance 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 (see 10.5.2 for jetted conductors) or piles drilled and grouted in place, the selection of skin friction values shall account for 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 over-consolidated clay, $f(z)$ may 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 addressed. A further check shall be made of the design value of the bond stress between the pile steel and the grout, as recommended in ISO 19902.

In layered soils, skin friction values, $f(z)$, in the clay layers shall be as given by Formula (22) to (24). End bearing values for piles tipped in clay layers with adjacent weaker layers can be as given in Formula (25) provided that:

a) the pile achieves penetration of two to three pile diameters or more into the clay layer, depending on the uncertainty on the depth of the top of the clay layer; and

b) the tip is approximately three pile diameters or more above the bottom of the clay layer to preclude punch-through.

Where these distances are not achieved, the design shall account for reduced end bearing.

For piles installed, partially or fully, in under-consolidated or creep-prone sediments, the potential for continuing consolidation or creep shall be assessed and any resulting negative skin friction addressed in estimating the available ultimate capacity and settlement of the piles.

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