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

Part 4: Geotechnical design considerations

Industries du pétrole et du gaz y compris les énergies à faible teneur en carbone — Exigences spécifiques relatives aux structures en mer —

Partie 4: Bases conceptuelles géotechniques

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

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

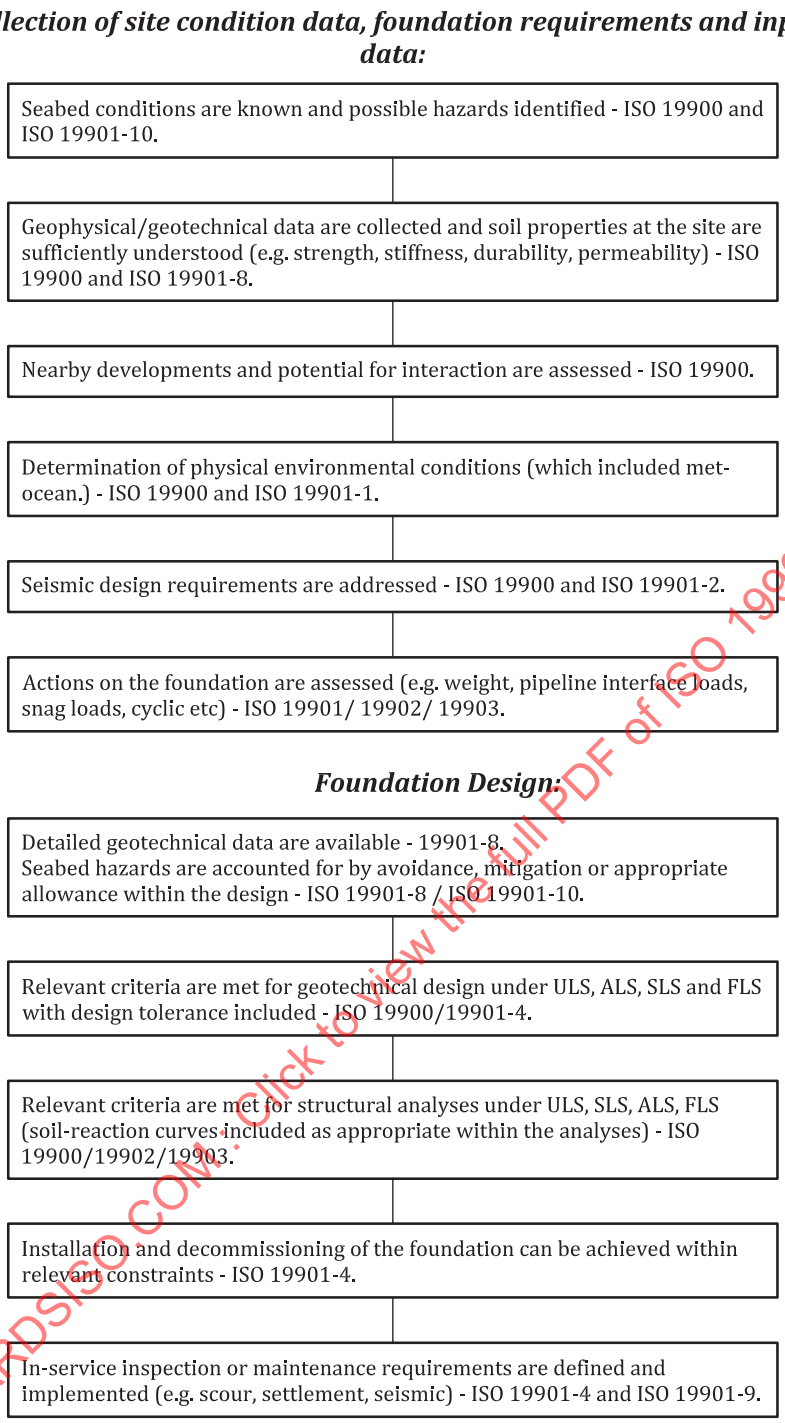
Some background to and guidance on the use of this document is provided in Annex A.

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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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 ≤ L/D ≤ 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 = πD

p_{mo} mobilised lateral soil resistance-displacement relationships under monotonic lateral loading (p_{mo}-y_{mo} curves)

p_r representative value of lateral capacity, for p–y curves, in unit of force per unit length of pile

p_{rd} representative value of deep lateral capacity, for p–y curves, in unit of force per unit length of pile

p_{rs} representative value of shallow lateral capacity, for p–y curves, in unit of force per unit length of pile

$p'_m(z)$ 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, $Q_{tn} = \left[\left(q_t - \sigma_{v0}\right)/p_a\right]\left(p_a - \sigma'_{v0}\right)^n$
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

$\gamma_{R,Pe}$ partial resistance factor for extreme conditions

$\gamma_{R,Pp}$ partial resistance factor for permanent and variable actions for operating situations

Ψ parameter to determine the dimensionless skin friction factor for clays at depth z, $\Psi = s_u(z) / \sigma'_{vo}(z)$

$\sigma'_{ho}(z)$ in situ effective horizontal stress at depth z

$\sigma_{vo}(z)$ in situ total vertical stress at depth z

$\sigma'_{vo}(z)$ in situ effective vertical stress at depth z

$\sigma'_{vo,tip}$ 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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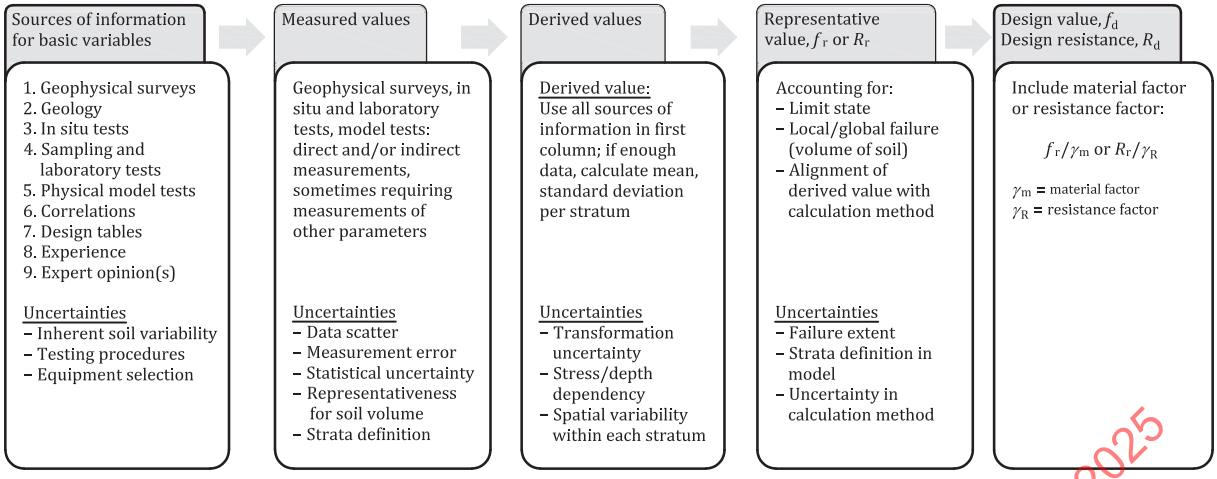


Figure 2 — Steps in selection of a representative value and design value for soil parameters

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.

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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.

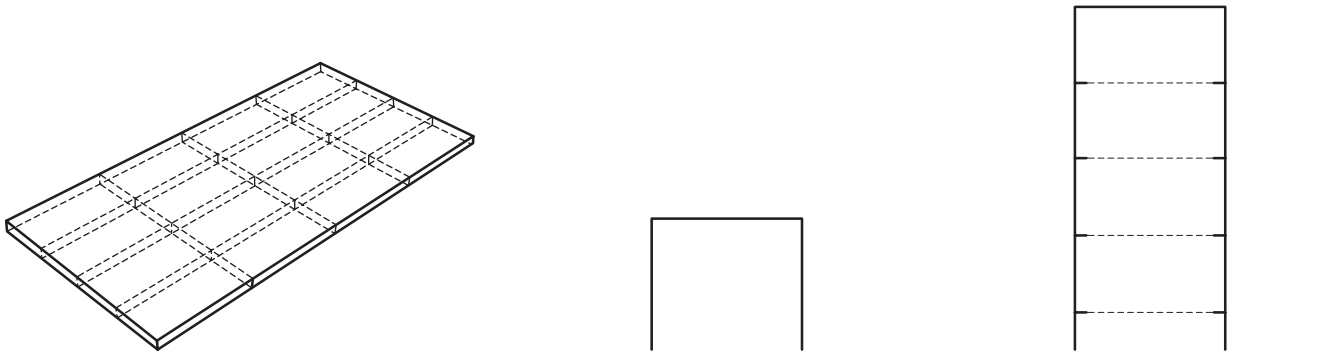
Internal skirts can be provided to increase stiffness of the foundation plate and prevent a soil failure mechanism developing within the soil plug. Stability assessment based on loading at skirt tip level shall verify that an internal failure mechanism does not form within the skirted compartment, which 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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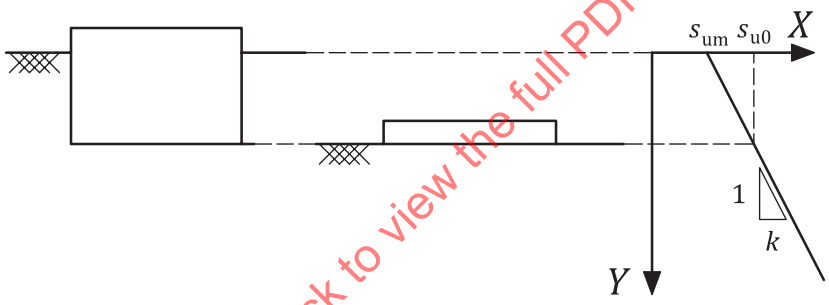
a) skirted shallow rectangular mud-mat with peripheral and internal skirts

(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.

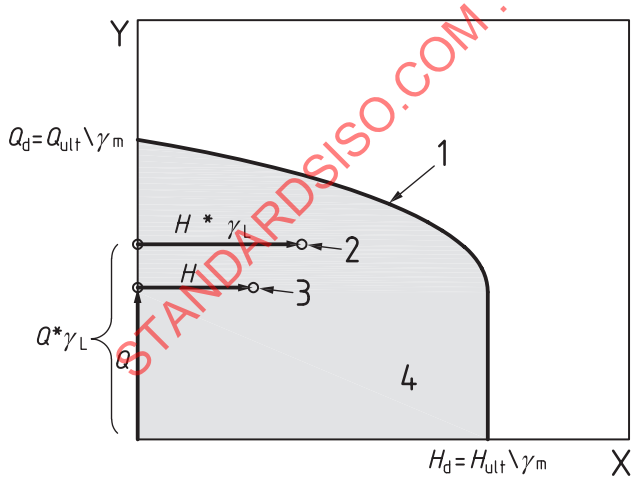
a) All action and material factors may be set to unity when calculating displacements as part of a serviceability assessment. However, where foundation displacement can lead to unacceptable consequences or governs design, factors greater than unity shall be used.

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.

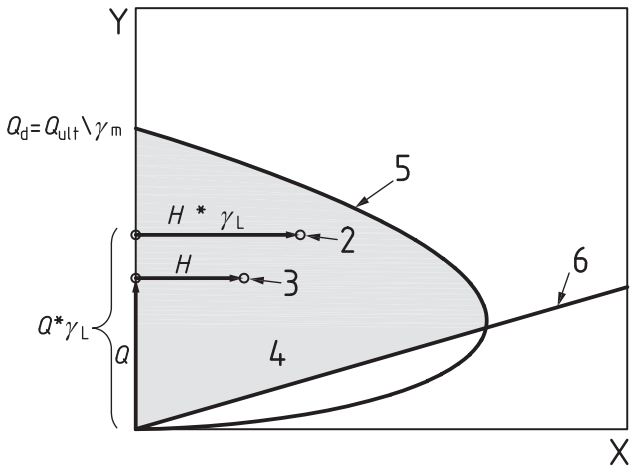
7.3.2 Use of partial factors in design

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:

$$q_d = N_c \frac{s_u}{\gamma_m} K_c \tag{2}$$

where

q_d is the design value of vertical bearing resistance, and note that $Q_d = q_d A'$;

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

b) Circular or square foundation:

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

7.5.1.5 Undrained conditions with linearly increasing shear strength with depth

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

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

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.

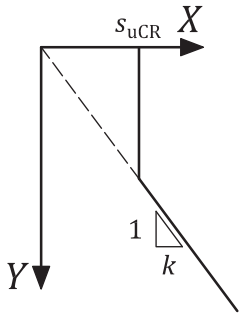
Details for calculation of F and K_c are provided in Clause A.7.

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:

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

b) horizontal:

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

c) overturning:

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

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;

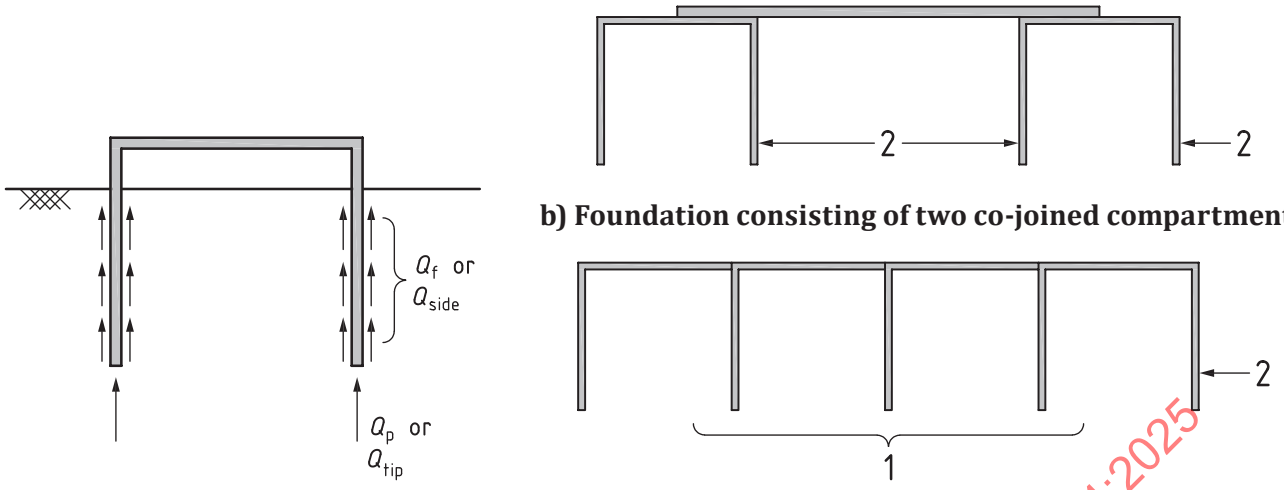
c) full displaced volume of internal skirts contributes to internal plug heave.

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:

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

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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8.1.4 Skin friction and end bearing in sands

This subclause presents the 'unified CPT method' for assessing pile capacity in sands [211]. Formulae (26) to (28) are presented for the evaluation of static capacity of steel tubular open-ended piles two weeks after driving. Piles are defined as foundations with an aspect ratio greater than 10. The use of the method for piles with aspect ratios less than 10 (e.g. monopiles) should be validated.

The method may be applied to sands with a fines content of less than 12 % (up to 20 % in the case of non-plastic fines). The method shall not be used for sands with unusually weak grains or compressible structure, including those sands containing amounts of mica, volcanic grains or calcium carbonate sufficient to lead to a mechanical response that differs from that of a silica sand, without further validation. Further guidance on other approaches to estimate pile capacity for sands that do not fall within the specified criteria is provided in A.8.1.4.

The unified CPT method shall only be applied to impact driven piles. The unified CPT method shall not be applied to vibro-driven piles, unless the vibrated portion of the pile is restricted to the first 20 % of pile penetration, within which the overall contribution to pile capacity is limited, unless proven otherwise. The unified CPT method shall not be applied to piles installed by jacking.

The reliability of the method has been evaluated and the method was shown, when used with the partial loading (action) and resistance factors of ISO 19902, to provide pile foundations that are more reliable than those obtained using the former main text method. The unified CPT method in sand may be used with the unified CPT method in clay, presented in A.8.1.3.2.2, to estimate the static capacity of driven piles in layered stratigraphy with a reliability that is essentially the same as that for cases where piles are installed in one soil type [40]. Further information on the method and Formulae (26) to (28) are provided in References [211] and [250].

In the absence of more definitive criteria, the external unit skin friction for capacity two weeks after driving, $f(z)$, in stress units, at depth, z, shall be calculated using Formula (26):

$$f(z) = f_L\,(\sigma'_{rc} + \Delta\sigma'_{rd})\tan 29^{o} \tag{26}$$

where

$$\sigma'_{rc} = \left(\frac{q_c}{44}\right) A_{re}^{0,3}\left[\mathrm{Max}\left[1, \left(\frac{h}{D}\right)\right]\right]^{-0,4}$$

$$\Delta\sigma'_{rd} = \left(\frac{q_c}{10}\right)\left(\frac{q_c}{\sigma'_v}\right)^{-0,33}\left(\frac{d_{ref}}{D}\right)$$

$$A_{re} = 1 - PLR\left(\frac{D_i}{D}\right)^2$$

where PLR=1,0 for typical offshore piles.

and:

f_L is a loading coefficient taken as 0,75 for tension actions and 1,0 for compression actions;

29° is the angle of interface friction used for calibration of the method, noting that factors, such as paint, coatings or mill-scale varnish, can negatively affect the interface friction that can be mobilized;

σ'_{rc} is the horizontal effective stress acting on a driven pile at a depth, z, about two weeks after driving;

σ'_v is the vertical effective stress at a depth, z;

q_c is the representative cone resistance at a depth, z;

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A_{re} is the effective area ratio, defined above, is a measure of the soil displacement induced by the driven pile and expressed as a fraction of the soil displacement induced by a closed-ended pile (for which A_{re}=1);

D is the pile outer diameter;

D_i is the pile inner diameter;

PLR is the plug length ratio, with a maximum value of 1,0, defined as the ratio of the plug length (L_p) to the pile embedment (L), and for which in the absence of measurements, PLR shall be taken as 1,0 for typical offshore piles;

$\Delta\sigma'_{rd}$ is the change in horizontal stress, acting a depth, z, arising due to interface shear dilation as the pile is loaded;

d_{ref} = 0,035 6 m;

h is the distance above the pile tip at which $f(z)$ acts (= L–z);

In the absence of more definitive criteria, the unit end bearing applied across the full base area of the pile shall be calculated using Formula (27):

$$q = [0{,}12 + 0{,}38 A_{re}]\, q_p \tag{27}$$

where

A_{re} is as defined per Formula (26);

q_p is a representative value of the CPT cone resistance near the pile tip, taking into account the following.

a) To allow for spatial variability, q_p is taken as the average q_c value within a zone 1,5D above and below the pile tip. However, because end bearing capacity can be sensitive to local variations in cone resistance, the impact of this sensitivity shall be reflected in the choice of representative value.

b) The end bearing method assumes a plugged base and is applicable for piles with a length to diameter ratio greater than five. For plugs with low permeability within two pile diameters of the pile tip, such as those comprising interbedded clay layers, the end bearing capacity can be reduced and in such cases, it shall be confirmed that unplugged end bearing is not less than plugged end bearing [235], [287]. In the absence of more definitive criteria, an estimate of unplugged end bearing can be taken according to Formula (28).

$$Q_p = q_{unplugged} A_{pile} \tag{28}$$

where

$$q_{unplugged} = A_{re} q_p .$$

The unified CPT method replaces the former main text method and the CPT methods previously defined in the annex. Further information on the latter CPT methods, experience with their application in practice and potential use in cases not covered by the unified CPT method can be found in [132], [177], [76] and [208]. For existing platforms where there are no CPT data available, the former main text method may still be used and is presented in A.8.1.4.3, although it is no longer recommended for calculation of pile capacity in sand.

8.1.5 Skin friction and end bearing in gravels

Characterizing the in situ density of gravel materials is difficult, with previous experience suggesting that site investigation results can be incorrectly interpreted, e.g. by characterizing the material as dense while pile driving can indicate a loose material.[183] The CPT based method presented in 8.1.4 and other pile design

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methods in Clause A.8, including the former main text method in sands in A.8.1.4.3, are likely to overpredict driven tubular pile capacity in gravels.[156] As a result, the methods presented in 8.1.4 and Clause A.8 shall not be used directly for pile design in gravels.

8.1.6 Skin friction and end bearing of grouted piles in rock

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

In general, the end bearing capacity is affected by and chosen depending on pile construction factors such as the degree of removal of drill cuttings from the base of the hole and on rock mass factors such as the presence of discontinuities within the rock mass. Unless these factors are addressed, the end bearing capacity shall be taken as zero.

In addition, the end bearing capacity of the rock shall not exceed the uniaxial compressive strength of the rock or grout multiplied by a bearing capacity factor appropriate for the type of rock. The limiting end bearing capacity for this type of pile can also be governed by stresses in the grout or in the pile steel.

Design values for (static) unit skin friction and end bearing can be found in various publications (e.g. References [197], [1] and [328]). Most publications on this subject refer to relatively 'stubby' stiff piles as used in onshore practice (bored piles). Owing to the brittle response applicable to unit skin friction, design values given in these publications can be unconservative for long flexible piles as used in offshore practice. In addition, the adverse effects of cyclic actions on the axial capacity of such piles shall be addressed. In cemented calcareous or carbonate material the skin friction assessment for grouted piles can be undertaken based on the load transfer (*t-z*) methodology and cyclic algorithm described in Reference [284]. The use of the design method and the assessment of the required design parameters to characterise the cemented material response shall be validated, e.g. with laboratory or field tests.

8.1.7 Skin friction and end bearing of driven piles in intermediate soils

In intermediate soils (e.g. silts or low plasticity silty tills), where cone penetration is usually partially drained, the applicability of the methods in 8.1.3 for clays and in 8.1.4 for sands shall be validated. Interpretations based on CPT resistance and piezocone response generally lead to higher shaft capacities when layers are deemed to be 'clays' rather than 'sands'. For example, the application of clay methods has led to significantly over-predicted axial capacity compared with offshore load tests with transitional soils whose grading curves span the range between the two soil types. In the absence of more definitive criteria, the designer can evaluate the degree of drainage observed during CPT profiling (ideally assisted with dissipation test data) and can evaluate the use of the minimum of shaft capacity estimates made by the sand and clay methods, as well as alternative procedures.

8.2 Pile capacity for axial tension

The representative value for pile axial pullout capacity, $Q_{r,t}$, is less than or equal to, but shall not exceed $Q_{f,c}$, the total skin friction capacity in compression. For clay soils, $f(z)$ shall be as stated in 8.1.3. For sands, $f(z)$ shall be computed in accordance with 8.1.4. For rock, $f(z)$ shall be assessed as stated in 8.1.6 and additionally shall address potentially lower pile capacity resulting from rock break-out mechanisms.

8.3 Axial pile performance

8.3.1 Static axial behaviour of piles

Pile axial deflections shall be within acceptable serviceability limits and these deflections shall be compatible with the internal forces and movements of the structure. The analysis procedure used shall simulate the nonlinear stress–strain behaviour of the soil. Displacements and rotations of individual piles shall satisfy serviceability limit states.

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In the absence of more definitive criteria, the relationships between mobilized axial shear transfer between pile and soil and the local pile displacement, and between mobilized end bearing resistance and the pile tip displacement, shall be determined in accordance with 8.4.

Axial pile behaviour is affected by directions, types, rates and sequence of the applied actions, by the installation technique, by soil type, by axial pile stiffness, as well as by other parameters. Some of these effects for clay soils have been observed in both laboratory and field tests.

In some circumstances (e.g. for soils that exhibit strain-softening behaviour), particularly where the piles are axially flexible, the actual axial capacity that can be mobilised by the pile can be less than that given by Formula (22), which assumes the pile is rigid. If *t–z* curves that exhibit strain-softening are recommended as per 8.4, the effect of axial pile flexibility on the pile capacity shall be addressed, either explicitly with t-z Q-z analyses or with approximate methods (e.g Referencce [274]). Other factors, such as increased axial capacity under loading rates associated with storm waves, can counteract these effects. More information is provided in the commentary in ISO 19902 as well as A.8.3.2 and Reference [67].

8.3.2 Cyclic axial behaviour of piles

Cyclic actions, including inertial actions due to environmental conditions such as storm waves and earthquakes, can have two potentially counteractive effects on the static axial capacity. Repetitive actions can cause a temporary or permanent decrease in resistance and an accumulation of deformation. Rapidly applied actions can cause an increase in resistance and stiffness of the pile. Very slowly applied actions can cause a decrease in resistance and stiffness of the pile. The resultant influence of cyclic actions is a function of the combined effects of the magnitudes, cycles and rates of change of applied actions, the structural characteristics of the pile and the types of soils (see A.8.3.2).

8.4 Soil reaction for piles under axial actions

8.4.1 Axial shear transfer *t–z* curves

The relationship between mobilized soil-pile shear transfer and local pile displacement at any depth is described using a *t–z* curve. Various empirical and theoretical methods are available for developing curves for axial shear transfer and pile displacement, *t–z* curves.

Resistance–displacement relationships for grouted piles are discussed in Reference [289], [279] and [290].

Curves developed from pile loading tests in representative soil profiles or based on laboratory soil tests that model pile installation can also be justified. In the absence of more definitive criteria, the *t–z* curves in Figure 8 shall be used for non-carbonate soils.

In clays, a typical value for z_{peak} of 1 % of the pile outer diameter (i.e. z_{peak}/D = 0,01) shall be used for routine design purposes. Values ranging from 0,25 % to 2,0 % may be used in cases where axial pile stiffness is critical for design.

In sands, in the absence of more definitive criteria, Formula (29), which provides a good fit to the unified database employed for derivation of Formula (26), shall be used for routine design of typical offshore piles[207].

$$\frac{z_{peak}}{D} = \frac{{q_c}^{0,5}{\sigma'_v}^{0,25}}{A p_a^{0,75}} \quad A = 1\,250\,(\text{comp}); A = 625\,(\text{tension}) \tag{29}$$

where

p_a is atmospheric pressure (100 kPa);

q_c is the representative CPT cone resistance;

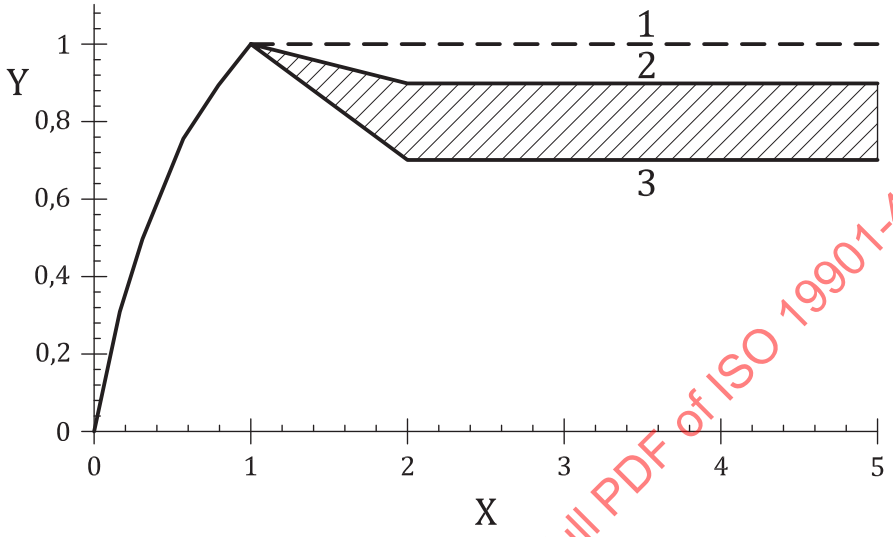
σ'_v is the vertical effective stress at the depth of the *t-z* spring.

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The empirical coefficient, A, depends on the loading direction and gives z_{peak}/D values in tension that are double those in compression.

The shape of the t–z curve at displacements greater than that at which t_{max} is reached as shown in Figure 11 should be assessed. Values of the residual friction ratio, t_{res}/t_{max}, and the axial pile displacement, z_{res}, at which it occurs, are a function of soil stress–strain behaviour, stress history, pile installation method, sequence of pile action application and other factors. Typical t_{res}/t_{max} values for clays range from 0,70 to 0,90; laboratory, in situ or model pile tests or local experience can provide valuable information for determining values of t_{res}/t_{max} and z_{res} for various soils.



z / z_{peak}	t / t_{max}	
	Clays	Sands
0	0	0
0,16	0,30	0,30
0,31	0,50	0,50
0,57	0,75	0,75
0,80	0,90	0,90
1,0	1,00	1,00
2,0	0,70 to 0,90	1,00
∞	0,70 to 0,90	1,00

Key

X z/z_{peak}
Y t/t_{max}
1 sand: t_{max}
2 clay: $t_{res} = 0{,}9\ t_{max}$
3 clay: $t_{res} = 0{,}7\ t_{max}$
z local pile axial displacement
z_{peak} displacement to maximum soil-pile unit skin friction
D pile outside diameter
t mobilised soil-pile unit skin friction (in stress units)
t_{max} = $f(z)$ = maximum soil-pile unit skin friction computed in accordance with 8.1 or 8.2 (in stress units)
t_{res} residual soil-pile unit skin friction (in stress units)
z_{res} axial pile displacement at which the residual soil-pile unit skin friction, t_{res}, is reached

Figure 8 — Typical axial pile shear transfer–displacement t–z curves in clays

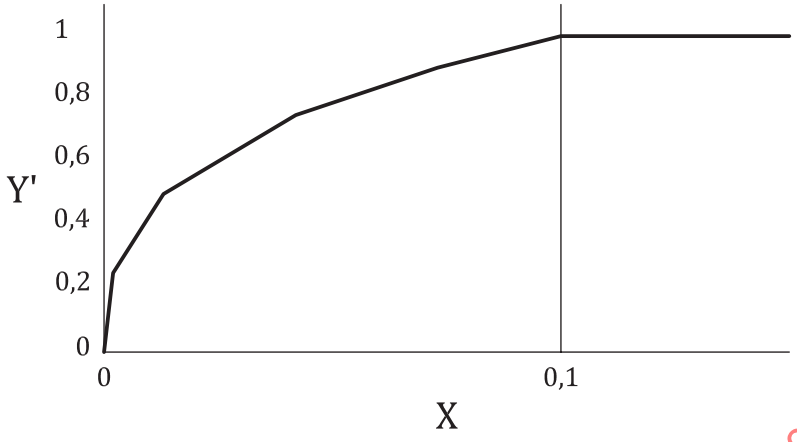
8.4.2 End bearing resistance–displacement, Q–z curve

Under compression actions, the relationship between mobilised end bearing resistance and axial pile tip displacement is described using a Q–z curve.

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A pile tip displacement of 10 % of the pile diameter can be required for full mobilization of the end bearing resistance described in 8.1 in both sand and clay soils. In the absence of more definitive criteria, the curve shown in Figure 9 shall be applied for both sands and clays.



z / D	Q / Q_p
0	0
0,002	0,25
0,013	0,50
0,042	0,75
0,073	0,90
0,100	1,00
∞	1,00

Key

X z/D
Y' Q/Qp
z axial pile tip displacement
D pile outside diameter
Q mobilised end bearing resistance (in force units)
Q_p representative value of end bearing resistance computed in accordance with 8.1 (in force units)

Figure 9 — Typical pile end bearing resistance–displacement *Q*–*z* curve

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8.5 Soil reaction for piles under lateral actions

8.5.1 General

The structural code of reference defines the lateral actions that the pile foundation shall be designed to resist. Such actions can include static, cyclic, fatigue, impact, and earthquake actions.

In the absence of more definitive criteria, the procedures given in 8.5.2 and 8.5.3 shall be used to construct the relationships between lateral soil resistance and lateral displacement, known as *p*–*y* curves. The development of these curves is documented in Reference [185]. The methods apply equally to all pile types, e.g. driven, jetted, and drilled and grouted piles.

The use of one-dimensional beam column analyses with only these p-y curves to model the soil reaction is applicable to long slender piles, i.e. piles with length to diameter, *L/D*, ratios greater than about 10. The analysis of intermediate foundations (e.g. short stubby piles such as large diameter piles with limited penetration) typically involves the use of additional soil reactions curves when using one dimensional beam-column techniques or different methods of analyses, as described in 8.5.6

The lateral resistance of the soil near the seafloor can be significant to pile design and the possible effects of seafloor scour on this resistance shall be addressed.

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In case of liquefied soils near the seafloor, the need to prevent pile buckling by reducing the critical compression stress for the applicable combinations of actions shall be addressed. See Reference [193].

8.5.2 Lateral soil reaction for clay

8.5.2.1 General

The framework within this clause is intended to provide a best estimate of the soil resistance. The framework was developed from numerical analyses combined with soil stress-strain behaviour, as measured in the laboratory through direct simple shear (DSS) tests performed at a standard shear strain rate of approximately 5 %/hr. DSS tests should be performed as per the guidance in ISO 19901-8. The undrained shear strength shall be taken as the maximum measured shear stress or the stress at a shear strain of 15 %, if no peak is observed.

It is applicable to soil consistencies ranging from very soft to very hard and has been validated by hindcast of pile load tests with soil strength values up to 600 kPa (12 531 lb/ft^2). It has not been validated for highly structured clays or carbonate materials.

The monotonic curves are derived as per 8.5.2.2 as a function of I_p and OCR, or from DSS testing.

The cyclic curves are derived as per 8.5.2.3 for three design conditions.

8.5.2.2 P-y curves for monotonic actions

8.5.2.2.1 General

The lateral failure mechanism of long slender piles consists of a wedge mechanism close to the seafloor and a flow-around mechanism at deeper depths.

For monotonic actions, the ultimate unit lateral resistance, p_u, in units of force per unit length of pile, has been found to vary between 9 s_u D and 12 s_u D for the flow-around mechanism. For depths where failure occurs with the wedge mechanism, the lateral capacity is reduced and depends on whether a gap is assumed to form on the back side of the pile.

P-y curves are developed by:

a) calculating p_u according to 8.5.2.2.2;

b) correcting p_u for anisotropy for gapping conditions, as per 8.5.2.2.3;

c) generating normalized *p-y* curves according to 8.5.2.2.4;

d) de-normalizing the *p-y* curves by using the values of p_u and D.

8.5.2.2.2 Ultimate soil resistance for isotropic conditions

In the absence of more definitive criteria, the ultimate soil resistance shall be calculated as:

$$p_u = P_u\, D \tag{30}$$

with

$$P_u = N_p\, s_u$$

$$N_p = N_{p0} + \frac{\gamma' z}{s_u} \le N_{pd} \quad \text{if gapping is assumed on the back side of the pile;}$$

$$N_p = 2N_{p0} \le N_{pd} \quad \text{if no gapping is assumed on the back side of the pile;}$$

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$$N_{p0} = N_1 - (1-\alpha_{ave}) - (N_1 - N_2)\left[1 - \left(\frac{z}{dD}\right)^{0,6}\right]^{1,35} \leq N_{pd}$$

$$N_1 = 12\ N_2 = 3,22$$

$$d = 16,8 - 2,3\log_{10}(\lambda) \geq 14,5$$

$$\lambda = s_{u0} / (s_{u1}\ D)$$

$$N_{pd} = 9 + 3\alpha_{ave}$$

where

d is a model parameter;

D is the pile outside diameter;

P_u is the ultimate soil pressure (in stress units);

p_u is the ultimate soil resistance per unit length (in units of force per unit length);

N_1 is a model parameter;

N_2 is a model parameter;

N_p is the total lateral bearing capacity factor;

N_{p0} is the lateral bearing capacity factor due to passive wedge for weightless soil;

N_{pd} is the lateral bearing capacity factor for flow around mechanism;

s_u is the representative undrained shear strength at the depth in question, as determined from DSS testing;

s_{u0} is the representative undrained shear strength at seafloor, as determined from DSS testing;

s_{u1} is the representative rate of increase of shear strength with depth in linearly increasing strength profiles, as determined from DSS testing. For moderately non-linear profiles, s_{u1} varies with depth and can be calculated as:

$$s_{u1}(z) = \frac{S_u(z) - S_{u0}}{z}$$

z is the depth below original seafloor;

α_{ave} is the average soil-pile skin friction factor, as calculated by Formulae (22) to (24), between the seafloor and a depth of 20 pile diameters, or over the pile length if L/D <20;

γ' is the soil submerged unit weight;

λ is the normalized rate of shear strength increase in linearly increasing shear strength profiles.

In the absence of DSS shear strength measurements, an equivalent DSS strength profile can be obtained from CPT, T-bar, UU, minivane or other measurements, based on local experience.

If a sand layer is present at the seafloor and overlays clay layers, the total bearing capacity factor N_p may be calculated as $N_p = N_{pd}$ at all depths in the clay layers if the sand layer is thicker than about one pile diameter.

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The choice of gapping condition, that is whether a gap is assumed on the back side of the pile or not, rests with the designer. The hindcast of 11 pile load tests suggests that pile behaviour (e.g. deflection, shear force, and bending moment profiles) are best predicted with the following assumptions when calculating N_p:

a) No gapping condition assumed in near-normally consolidated profiles where the shear strength is less than about 15 kPa in the top 10 m, regardless of the magnitude of lateral displacements.

b) Gapping condition assumed at large lateral displacement if the shear strength is less than about 15 kPa at the seafloor and exceeds 15 kPa within the top 10 m. For such strength profiles, no gapping conditions can be assumed at small lateral displacements and gapping condition can be assumed for large lateral displacements.

c) Gapping condition assumed at all values of lateral displacements if the shear strength is greater than 15 kPa at all depths in the top 10 m.

Differences between the above pile load tests and a typical offshore design case can include the presence of mudmats, or other seafloor structures surrounding the piles, which can confine soils and prevent gapping.

8.5.2.2.3 Anisotropy correction of ultimate soil resistance for gapping conditions

The formulae of 8.5.2.2.2 were derived for isotropic conditions. In the wedge failure mechanism, i.e. if $N_p < N_{pd}$, and for gapping condition only, the strength measured in triaxial extension should be used instead of the DSS strength. In absence of more definitive criteria, where gapping is assumed on the back side of the pile, the bearing capacity factor N_p shall be corrected for anisotropy as follows:

$$N_{p_{cor}} = C_w \, N_{p0} + \frac{\gamma' z}{s_u} \leq N_{pd} \tag{31}$$

with:

$$C_w = 1 + \left(\frac{s_{uTE}}{s_{uDSS}} - 1 \right) \left(\frac{N_{pd} - N_p}{N_{pd} - N_p|_{z=0}} \right)$$

where

N_{p_cor} is the total lateral bearing capacity factor corrected for anisotropy;

C_w is the anisotropy correction factor for the wedge failure mechanism for gapping conditions;

$\frac{s_{uTE}}{s_{uDSS}}$ is the average ratio of triaxial extension strength over DSS strength within the depth of the wedge, for which in the absence of site-specific data, a default value of 0,9 is recommended for very soft and soft clays like those of the Gulf of Mexico;

N_p is the total lateral bearing capacity factor for isotropic conditions, as per 8.5.2.2.2;

N_{po} is the lateral bearing capacity factor due to passive wedge for weightless soil for isotropic conditions, as per 8.5.2.2.2;

N_{pd} is the lateral bearing capacity factor for flow around mechanism for isotropic conditions, as per 8.5.2.2.2;

$N_p|_{z=0}$ is the value of N_p for isotropic conditions, as per 8.5.2.2.2, at the original seafloor elevation (i.e. $z = 0$).

Further guidance on soil anisotropy and soil profiles with seafloor crusts can be found in A.8.5.2.1.2.

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8.5.2.2.4 *P–y* curve relationships

Lateral soil resistance–displacement relationships for piles in clays are nonlinear. The *p–y* curves for monotonic actions may be generated by scaling the stress-strain curves measured in the laboratory through DSS testing by either of the two methods presented in A.8.5.2.2.3.

Alternatively, the monotonic *p-y* curves ($p_{mo} - y_{mo}$) shall be generated from the default normalized curves of Table 1, as plotted in Figure 10.

Table 1 — Normalized p-y curves for monotonic actions for clay

	$I_p > 30$ %			$I_p \leq 30$ %		
	OCR ≤ *2*	OCR = *4*	OCR = *10*	OCR ≤ *2*	OCR = *4*	OCR = *10*
p_{mo} / p_u	y_{mo} / D	y_{mo} / D	y_{mo} / D	y_{mo} / D	y_{mo} / D	y_{mo} / D
0	0	0	0	0	0	0
0,05	0,000 3	0,000 4	0,000 5	0,000 1	0,000 2	0,000 3
0,2	0,003	0,004	0,005	0,001	0,002	0,003 3
0,3	0,005 3	0,008	0,011	0,001 8	0,004	0,007 3
0,4	0,009	0,015	0,021	0,003	0,007 5	0,014
0,5	0,014	0,024	0,034	0,004 8	0,012	0,023
0,6	0,022	0,036	0,052	0,007 3	0,018	0,035
0,7	0,032	0,055	0,078	0,011	0,027	0,052
0,8	0,05	0,084	0,12	0,017	0,042	0,08
0,9	0,082	0,14	0,19	0,027	0,07	0,13
0,975	0,15	0,23	0,3	0,05	0,11	0,2
1,0	0,25	0,3	0,4	0,083	0,15	0,27
1,0	∞	∞	∞	∞	∞	∞

Key

p_{mo} / p_u : normalized lateral soil resistance for monotonic actions

y_{mo} / D : normalized lateral displacement for monotonic actions

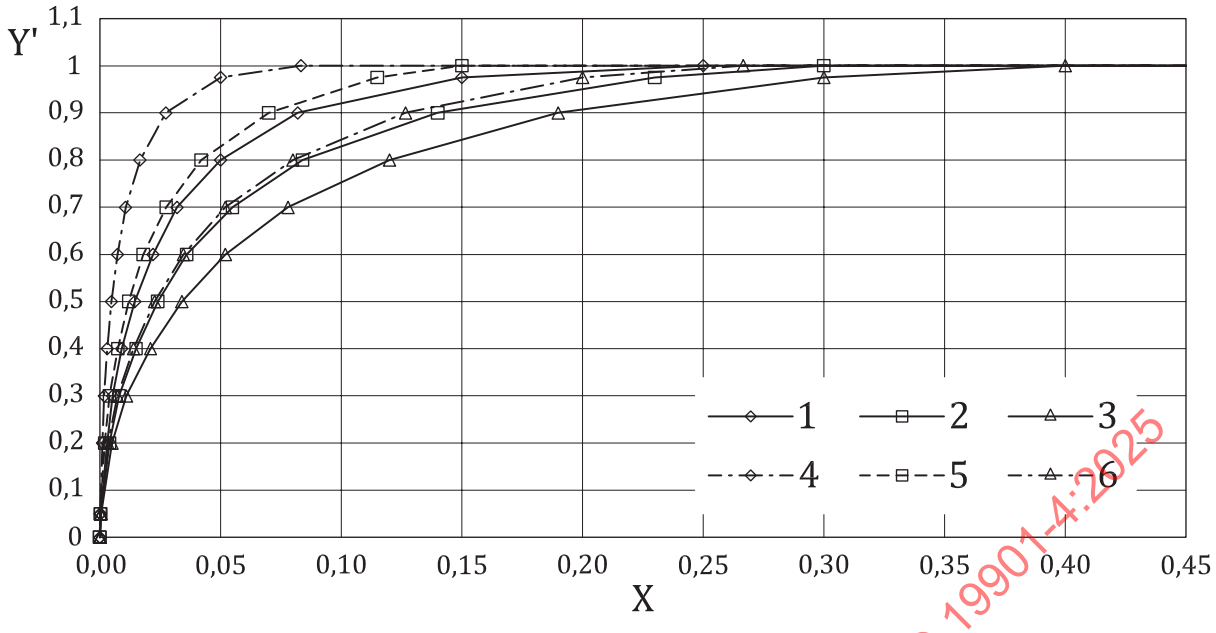
OCR: over-consolidation ratio

I_p: plasticity index

All other variables as defined in 8.5.2.2.2

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Key

X	normalized lateral displacement, (y/D)	3	I_p > 30 %; OCR = 10
Y'	normalized resistance, (p/p_u)	4	I_p ≤ 30 %; OCR ≤ 2
1	I_p > 30 %; OCR ≤ 2	5	I_p ≤ 30 %; OCR = 4
2	I_p > 30 %; OCR = 4	6	I_p ≤ 30 %; OCR = 10

Figure 10 — Normalized *p-y* curves for monotonic actions for clay

8.5.2.3 *P-y* curves for cyclic actions

The procedure may be applied to three defined design conditions.

— Gulf of Mexico (GoM) conditions: Piles in clays with OCR less than 2,0, with normalized cyclic shear strength properties like those of the GoM clays and subjected to loading conditions like those experienced by piles on fixed structures in the GoM.

— North Sea soft clay conditions: Piles in clays with OCR less than 2,0, with normalized cyclic shear strength properties like those of the Drammen clay and subjected to loading conditions like those experienced by piles on fixed structures in the North Sea.

— North Sea stiff clay conditions: Piles in clays with OCR greater than 4,0, with normalized cyclic shear strength properties like those of the Drammen clay and subjected to loading conditions like those experienced by piles on fixed structures in the North Sea.

In the absence of more definitive criteria, the cyclic *p-y* curves ($p_{cy} - y_{cy}$) used to calculate pile behaviour under the maximum storm loading shall be calculated as follows:

$$p_{cy} = p_{mod}\ p_{mo} \tag{32}$$

$$y_{cy} = y_{mod}\ y_{mo} \tag{33}$$

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where

p_{cy} is the lateral soil resistance for cyclic actions;

y_{cy} is the lateral displacement for cyclic actions;

p_{mod} is the *p*-modifier model parameter;

y_{mod} is the *y*-modifier model parameter.

The *p*-modifer and *y*-modifer shall be calculated according to the following steps:

— Determine the depth of rotation of the pile, z_{rot}, under the peak total lateral action.

The depth of rotation of the pile is defined as the first depth from the seafloor where the lateral displacement is zero.

If the action, pile diameter, and pile wall thickness schedule are known, the depth of rotation may be determined by a beam column analysis using the *p-y* springs calculated for monotonic actions, as per 8.5.2.2.

Alternatively, the depth of rotation of the pile shall be estimated as $z_{rot} = 15D$.

— Calculate the hybrid factor, h_f, at each depth z and for all points p_{mo} / p_u on the normalized monotonic *p-y* curves:

$$h_f = \frac{p_{mo}}{p_u} - \left(\frac{z}{z_{rot}} \right)^2 \quad \text{if } z \le z_{rot}$$

$$h_f = \frac{p_{mo}}{p_u} - 1 \quad \text{if } z > z_{rot}$$

where

h_f is a model parameter and $-1 \le h_f \le +1$.

— Calculate the number of equivalent cycles, N_{eq}, at each depth z and for all points p_{mo} / p_u on the normalized monotonic *p-y* curves:

$$N_{eq} = \left(\frac{2}{1 - h_f} \right)^g \le 25$$

where

N_{eq} is a model parameter and $1 \le N_{eq} \le 25$;

g is a model parameter and:
g = 1,0 for Gulf of Mexico conditions;
g = 1,25 for North Sea soft clay conditions;
g = 2,5 for North Sea stiff clay conditions.

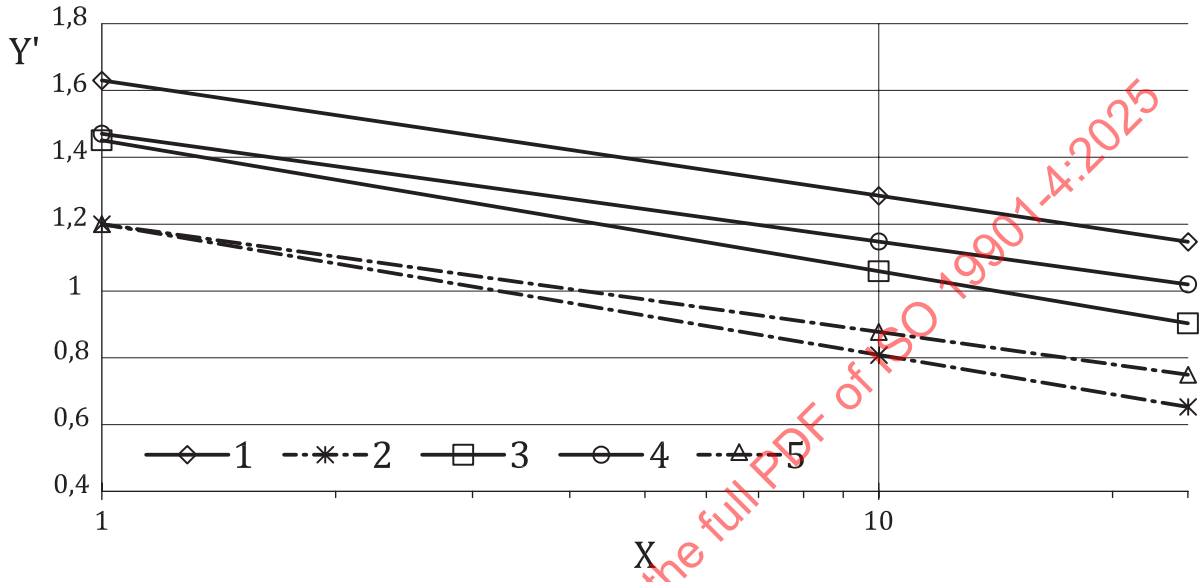
— Calculate the *p*-modifier and the *y*-modifier at each depth z and for all points p_{mo} / p_u on the normalized monotonic *p-y* curves as per Table 2, as plotted on Figure 11.

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Table 2 — Cyclic modifiers for *p-y* curves in clays

Design condition	Cyclic modifiers	
Gulf of Mexico	$p_{mod} = 1{,}47 - 0{,}14 \ln(N_{eq})$	$y_{mod} = 1{,}2 - 0{,}14 \ln(N_{eq})$
North Sea soft clay	$p_{mod} = 1{,}63 - 0{,}15 \ln(N_{eq})$	$y_{mod} = 1{,}2 - 0{,}17 \ln(N_{eq})$
North Sea stiff clay	$p_{mod} = 1{,}45 - 0{,}17 \ln(N_{eq})$	$y_{mod} = 1{,}2 - 0{,}17 \ln(N_{eq})$



Key

X number of equivalent cycles, N_{eq} (-)
Y' *P*-modifier, P_{mod}, or *y*-modifier, Y_{mod}
1 P_{mod} - North Sea soft clay
2 Y_{mod} - North Sea soft and stiff clay
3 P_{mod} - North Sea stiff clay
4 P_{mod} - Gulf of Mexico
5 Y_{mod} - Gulf of Mexico

Figure 11 — Normalized *p-y* curves for monotonic actions for clay

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8.5.2.4 *P-y* curves for fatigue actions

Fatigue actions typically cause a very large number of cycles of very low pile displacements. The curves for fatigue analyses (p_{fa} - y_{fa}) represent the steady-state conditions after the soil unload-reload secant stiffness and hysteretic damping have stabilized, typically after several hundreds of cycles.

A spring-only model is presented for piles for both static and dynamic, e.g. time-domain, analyses. A spring-dashpot model is presented in A.10.5.5.2.2 for conductors supporting typical subsea wellhead/BOP/LMRP systems, but is not applicable to piles. At very low lateral displacements, the fatigue curves are stiffer than the cyclic curves because the stiffness of the spring-only model includes the effect of hysteretic damping. Further guidance is provided in A.8.5.2.4.

In the absence of more definitive criteria, the (p_{fa} - y_{fa}) curves shall be calculated as:

$$p_{fa} = p_u \, A_s \cdot \left(\frac{y_{fa}}{D} \right)^{-B_s} \tag{34}$$

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where

p_{fa} is the lateral soil resistance for fatigue actions;

y_{fa} is the lateral displacement for fatigue actions;

p_u is the ultimate soil resistance calculated as per 8.5.2.2.2, with the no-gapping condition;

A_s is a model parameter; A_s = 0,45 if s_u < 40 kPa and A_s = 0,19 for other conditions;

B_s is a model parameter; B_s = 0,05 for all conditions.

The power-law curve of Formula (34) is discretized as follows:

a) the curve shall pass through the origin $y_{fa} = p_{fa} = 0$;

b) the next point shall be generated for y_{fa}/D=0,001;

c) the following points should be generated at the following y_{fa}/D increments, up to a maximum y_{fa}/D = 0,05:

d) 0,001 for 0,001 < $y_{fa}/D \leq$ 0,010;

e) 0,01 for 0,010 < $y_{fa}/D \leq$ 0,05.

8.5.2.5 *P-y* curves for earthquake actions

The monotonic curves of 8.5.2.2 can be used to analyse piles under seismic actions. They have shown to provide a satisfactory match between measured and calculated bending moments in fixed-structures piles when they are combined with appropriate unload-reload behaviour and parallel dashpots to model radiation damping. Further guidance is provided in A.8.5.2.5.

The curves for cyclic actions of 8.5.2.3 have been developed for three specific metocean action conditions and not for earthquake actions. Their use for earthquake conditions shall be validated.

8.5.2.6 Comparison with previous recommendations

When compared with the *p-y* curve methods in Reference [174], the recommended practice of 8.5.2 generally gives *p-y* curves that are stiffer at low displacements and with greater ultimate resistance.

Comparisons between the curves obtained by previous and current recommendations are provided in A.8.5.2.6.

8.5.3 Lateral capacity for sand

For static lateral actions, the representative unit lateral capacity, p_r, for sand has been found to vary from a value at shallow depths determined by Formula (35) to a value at deep depths determined by Formula (36). In the absence of more definitive criteria, at a given depth, the formula giving the smallest value of p_r shall be used as the representative capacity. These formulae can be un-conservative for layered soil conditions when the sand is overlain by soft clay.

$$p_{rs} = (C_1 z + C_2 D)\gamma' z \quad (35)$$

$$p_{rd} = C_3 D \gamma' z \quad (36)$$

where 's' signifies shallow and 'd' signifies deep, and

D is the pile outside diameter;

p_r is the representative lateral capacity (in force per unit length of pile);

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γ' is the submerged unit weight of soil (kN/m^3);

z is the depth below original seafloor (m);

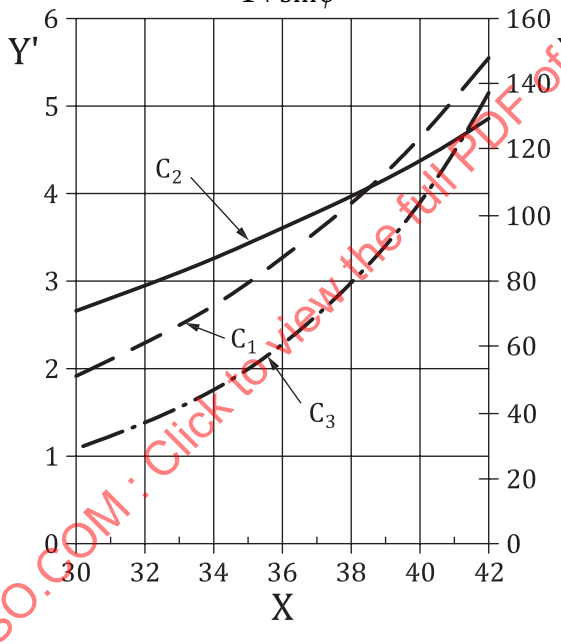
C_1, C_2, C_3 are dimensionless coefficients as a function of the effective angle of internal friction in sand, ϕ' (see Figure 12).

$$C_1 = \frac{(\tan\beta)^2 \tan\alpha}{\tan(\beta-\phi')} + K_0\left(\frac{\tan\phi'\sin\beta}{\cos\alpha\tan(\beta-\phi')} + \tan\beta(\tan\phi'\sin\beta - \tan\alpha)\right) \quad (37)$$

$$C_2 = \frac{\tan\beta}{\tan(\beta-\phi')} - K_a \quad (38)$$

$$C_3 = K_a\left((\tan\beta)^8 - 1\right) + K_0\tan\phi'(\tan\beta)^4 \quad (39)$$

with $\alpha = \frac{\phi'}{2}$; $\beta = 45 + \frac{\phi'}{2}$; $K_0 = 0{,}4$ and $K_a = \frac{1-\sin\phi'}{1+\sin\phi'}$



Key

X	effective angle of internal friction (ϕ' in degrees)	C_1	coefficient C_1 for lateral capacity
Y'	value of coefficients C_1 and C_2	C_2	coefficient C_2 for lateral capacity
Y''	value of coefficient C_3	C_3	coefficient C_3 for lateral capacity

Figure 12 — Lateral capacity coefficients for sand

Guidance for scour conditions is given in A.8.5.3.

8.5.4 Lateral soil resistance – displacement *p–y* curves for sand

The lateral soil resistance–displacement p–y relationship for a pile in sand is also nonlinear and in the absence of more criteria, at any specific depth, z, shall be determined by:

$$p = A\,p_r \times \tanh\left(\frac{k\,z}{A\,p_r}\,y\right) \quad (40)$$

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where

A is a factor to account for static or cyclic actions, evaluated by

$A = \left(3{,}0 - \frac{0{,}8.z}{D}\right) > 0{,}9$ for static actions, and

A = 0,9 for cyclic actions;

p_r is the representative lateral capacity at depth z (force per unit length of pile);

k is the initial modulus of subgrade reaction (force per volume), see Table 3;

z is the depth below original seafloor (m);

y is the lateral displacement at depth z.

The database for the lateral soil-pile behaviour in sands consists of free-head tests on piles in clean sands, with effective angles of internal friction ranging from 34° to 42°, as determined by shear box tests, drained triaxial tests or correlations with in situ tests.

Extrapolation of these data to soils outside the limits of experience, particularly to those sands with effective angles of internal friction less than 30°, requires engineering judgement. In particular, laboratory test results on such soils should be reviewed for evidence of anomalous behaviour and for the presence of significant fractions of clay soils, either of which could require a different formulation for the p–y relationships.

In the absence of more definitive criteria, the values of the initial modulus of subgrade reaction, k, given in Table 3 shall be applied.

Table 3 — Initial modulus of subgrade reaction

ϕ'	k
	MN/m^3
25°	5,4
30°	8,7
35°	22
40°	45

8.5.5 *p–y* curves for fatigue actions for sands

Similar to the recommendations for clays, the curves for fatigue analyses (p_{fa} - y_{fa}) represent the steady-state conditions after the soil unload-reload secant stiffness and hysteretic damping have stabilized, typically after several hundreds of cycles. A spring-only model is presented for piles, to be used for both static and dynamic (e.g. time-domain) analyses.

In the absence of more definitive criteria, the (p_{fa} - y_{fa}) springs shall be calculated as:

$$p_{fa} = 365\, D\, \tau(z) \left(\frac{y_{fa}}{D}\right)^{0{,}65} \tag{41}$$

where

p_{fa} is the lateral soil resistance for fatigue actions;

y_{fa} is the lateral displacement for fatigue actions;

$\tau(z)$ is the shear resistance. $\tau(z) = \sigma'_{vo}(z)\tan(\phi')$.

The power-law curve of Formula (41) is discretized as per 8.5.2.4.

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8.5.6 Refined assessment of lateral pile response

A more advanced one-dimensional pile analysis method, which involves the definition of case-specific soil reaction curves, is set out for applications where the design is either highly sensitive to lateral actions or when the pile L/D ratio is low [59][63]. The method's development is further described in A.8.5.6.

8.5.7 Lateral soil resistance-displacement curves in calcareous soil, cemented soil and weak rock

The degradation of strength and stiffness of cemented or calcareous soil and weak rock materials under cyclic actions shall be addressed in the design of grouted pile and conductor (see Reference [205]). The results from site-specific pile test and centrifuge model test data, where available, may also be used to calibrate site-specific *p-y* data.

8.6 Pile group behaviour

8.6.1 General

The effects of closely spaced adjacent piles on the resistance–displacement characteristics of the pile group should be considered. For pile spacing less than eight diameters center-to-center, group effects shall be addressed in lateral response design.

The pile group axial capacity shall conform to the requirements of 8.1.1. Where there is a non-uniform distribution of actions into the piles, the partial resistance factors for individual piles axial capacity in the group may be less than those specified in 8.1.1, provided it is demonstrated that the displacements and corresponding deformations and stresses of the piles and associated structural members are acceptable.

8.6.2 Axial behaviour

For piles embedded in clays, the group capacity can be less than the single isolated pile capacity multiplied by the number of piles in the group; conversely, for piles embedded in sands, the group capacity can be higher than the sum of the capacities of the isolated piles. The group settlement in either clay or sand is normally larger than that of a single pile subjected to the average action per pile of the pile group.

8.6.3 Lateral behaviour

For piles with the same pile head fixity conditions and which are embedded in either clay or sands, the pile group normally experiences greater lateral displacements than those undergone by a single pile subjected to the average action per pile of the corresponding group. The major factors influencing the group displacements and distribution of actions over the piles are the direction of loading, the pile spacing, the ratio of the pile penetration to the pile diameter, the pile flexibility relative to the soil, the dimensions of the group, and the variations in the shear strength and stiffness modulus of the soil with depth.

Of the four group analysis methods examined in Reference [256], the following methods were found to be the most appropriate for use in designing group pile foundations for the given actions:

a) for defining initial group stiffness: advanced methods, such as PILGP2R [256];

b) for design event actions: the Focht-Koch method [128] as modified by Reese et al. [290] for defining group displacements and average maximum pile moments; displacements are probably underpredicted at actions giving displacements of 20 % or more of the diameter of the individual piles in the group;

c) for evaluating maximum pile action at a given group displacement: largest value obtained from the original or modified Focht-Koch method.

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8.7 Pile installation assessment

8.7.1 General

The types of pile foundations used to support offshore structures and considered in this document are as follows.

a) Driven piles: Open-ended piles are normally used in foundations for offshore structures (see 8.7.2 to 8.7.7). These piles are usually driven into the seabed with impact hammers, which use steam, diesel fuel or hydraulic power as the source of energy.

b) Drilled and grouted piles: Piles that can be used in soils and rocks which will hold an open hole with or without drilling mud (see 8.7.8).

c) Vibro-driven piles: The capability of hydraulic vibratory driving hammers to install piles has been demonstrated, in particular for the installation of small diameter piles in sands. Limited data are available with respect to the effect of the installation method on the pile axial capacity and the methods of 8.1. should be validated.

The pile wall thickness and driving shoe shall be sufficient to resist axial and lateral actions as well as the stresses during pile installation. The pile stresses and, as a result thereof, the minimum pile wall thickness shall also conform to the requirements from ISO 19902 where the pile strength shall be verified using the steel tubular checking formulae given in ISO 19902 for conditions of combined axial force and bending. Fatigue damage due to pile driving shall be accounted for when calculating the pile fatigue life in accordance with ISO 19902.

Damage to the pile tip during offshore handling and installation can lead to localised pile buckling during driving. The potential for global or local pile buckling shall be assessed. If used, the effect of the driving shoe shall be addressed in the choice of design skin friction and end bearing values (driveability and axial capacity).

All field-made structural connections shall be compatible with the design requirements. Pile sections should be marked to facilitate installing the pile sections in the planned sequence. The closure device on the lower end of the structure's legs and pile sleeves, if required, shall be designed to avoid interference with the installation of the piles.

8.7.2 Drivability studies

Computer analyses (based on the principles of one-dimensional elastic stress-wave theories and commonly known as wave equation analyses) may be used to simulate the hammer-pile-soil system and pile driving behaviour, with the objective of defining the range of blow counts necessary to reach the target design pile penetration and assessing the stresses in the pile resulting from pile driving. The predicted range of blow counts to reach a given penetration is governed by the estimated profile of soil resistance to driving (SRD), by the assumed hammer efficiency, or driving energy transferred to the pile and, to a lesser extent, by the quake and damping parameters in the wave equation model. Selection of these input parameters is based on previous pile driving experience and engineering judgment. For a given rated energy, the energy transferred to the pile is dependent on the type of hammer (i.e. diesel fuel, steam, or hydraulic hammer) and is based on pile driving experience with reliable measurements from pile instrumentation.

The calculation method for the SRD is the main factor that governs the results of the drivability studies and the hammer type required to reach the target pile penetration. Several methods for calculating the SRD in different types of soils have been proposed in the literature (see A.8.7.2).

General procedures cannot be applied at all sites, as piling behaviour is site dependent. Therefore, back-analysis of previous pile driving experience at the site, or at sites with similar soil conditions, should be performed in order to calibrate SRD calculation procedures and improve drivability predictions for other structures at the site. For clays, the SRD calculation should account for the increase in resistance due to pore pressure dissipation (set-up) during driving interruptions (e.g. when delays are necessary for welding pile add-on elements).

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To confirm that the hammer performs in accordance with the specifications and with the assumptions made in the drivability predictions, the pile or hammer may be instrumented and monitored during driving. If so, pile instrumentation is preferable, as hammer monitoring provides incomplete information about the driving energy transferred into the pile.

Pile driving instrumentation data, based on measurements from strain and acceleration transducers fixed near the top of the pile, may be used for verifying the actual hammer driving energy and soil stratification, assessing the actual SRD during driving, as well as giving additional information for estimating the pile capacity, particularly if re-strike test data are available. The SRD measured during driving, as back-calculated from pile instrumentation data, should be compared with the predicted range in soil resistance. Such analyses can be used to improve the reliability of subsequent drivability predictions at the site.

Selection of representative parameters or methodology for driveability analyses, or driveability input to fatigue analyses, should account for the experience in the area or in similar soil conditions, with upper estimate parameters being used if experience does not exist or is limited and best-estimate parameters being used otherwise.

8.7.3 Obtaining required pile penetration

The adequacy of the structure's foundation depends upon each pile being driven to or near its design penetration. Where applicable, the driving of each pile should be carried to completion with as little interruption as possible to minimize the increased driving resistance which often develops during delays. Workable back-up hammers with leads should be available, especially when critical pile set-up is anticipated.

The fact that a pile has met premature refusal does not indicate that it can support the design actions. Final blow count alone shall not be considered as assurance of piling adequacy.

In some instances, when continued driving is not successful, the penetration and associated capacity of a pile can be improved by the methods described in 8.7.5.

8.7.4 Driven pile refusal

Pile refusal is defined to:

a) establish the point at which pile driving with a particular hammer should be stopped and other methods instituted (see 8.7.5); and

b) prevent damage to the pile or hammer.

The definition of refusal should be consistent with the soil characteristics anticipated at the specific location. Refusal should be defined for all hammer sizes to be used and is contingent upon the hammer being operated at the energy and rate recommended by the manufacturer.

The exact definition of pile refusal for a particular installation should be defined in the installation specification. Examples of refusal criteria, for use only if no other requirements are included in the installation specification, are given in A.8.7.4.

If a pile refuses before it reaches design penetration, one or more of the measures given in 8.7.5 may be taken.

8.7.5 Pile refusal remedial measures

8.7.5.1 Review of hammer performance

A review of all aspects of hammer performance, possibly with the aid of hammer and/or pile head instrumentation, can identify problems that can be solved by improved hammer operation and maintenance, or by the use of a more powerful hammer.

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8.7.5.2 Re-evaluation of design penetration

Reconsideration of actions, displacements and required capacities of individual piles, of other foundation elements and of the foundation system, can identify available reserve capacity.

An interpretation of driving records in conjunction with instrumentation can allow the design soil parameters or stratification to be revised and the calculated pile capacity to be revised.

8.7.5.3 Modifications to piling procedures

8.7.5.3.1 General

Modifying procedures, can permit the piles to be driven to the required penetration. The modifications described in 8.7.5.3.2 to 8.7.5.3.4 may be used.

8.7.5.3.2 Plug removal

The soil plug inside the pile can be removed by jetting and air lifting, or by drilling, to reduce pile driving resistance. Several soil plug removals and redrives can be required to reach target penetration.

If plug removal results in inadequate pile capacity, the removed soil plug shall be replaced by a grout or concrete plug or a plug made from another suitable material to increase the capacity back to the design level. The minimum axial capacity of the plug shall be equal to the pile end bearing capacity in a plugged condition, if full shear transfer between plug and pile is available. In some circumstances plug removal is not effective in improving driving conditions, particularly in cohesive soils.

8.7.5.3.3 Soil removal below the pile tip

Soil below the pile tip can be removed, either by drilling an undersized hole or by jetting and possibly air lifting. The drilling or jetting equipment is lowered through the pile, which acts as the casing pipe for the operation. Considering the resulting uncertainties with respect to the pile axial capacity, the soil below the pile tip should not be removed to reduce the soil resistance during driving in uncemented soils.

Under special circumstances (e.g. in the case of an intermediate layer of strong cemented material), undersized drilling can be applied to partially remove the hard layer before pile driving can be resumed. The depth of drilling should be restricted to the thickness of the hard-cemented layer.

Undersized drilling should be restricted to relatively thin and not too hard layers. In thick and hard rock layers under-reaming of the hole to at least the full pile size should be evaluated to avoid potential risk of pile tip buckling.

Where soil removal below the pile tip has been performed by drilling (undersized or otherwise), the contribution of the relevant zone of soil to the pile capacity should be ignored, unless this zone has been grouted.

Jetting below the pile tip should be avoided because of the unpredictability of the results.

8.7.5.3.4 Two-stage driven piles

A first-stage or outer pile can be driven to a predetermined depth, after which the soil plug is removed and a second-stage or inner pile is driven inside the first-stage pile. The annulus between the two piles is grouted to permit shear transfer between the first- and second-stage piles and to develop composite action.

8.7.6 Selection of pile hammer and stresses during driving

The influence of the hammers to be used shall be evaluated as part of the design process in accordance with ISO 19902 for the definition of pile wall thickness and stresses generated by hammer placement and pile driving. A method of analysis based on wave propagation theory shall be used to determine the dynamic stresses generated by hammer impact.

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The type(s) of pile hammer considered for pile driving shall be noted on the installation drawings or specifications. Any change in the hammers to be used for pile driving shall be assessed, to verify that the consequences of the change are acceptable, including pile drivability, pile capacity, pile and structure strength and fatigue. Detailed guidance is provided in ISO 19902.

Items relevant to pile design and installation assessment are as follows.

a) Stresses during driving: The unfactored dynamic stresses should not exceed 90 % of yield, depending on specific circumstances such as the location of the maximum stresses down the length of pile, the number of blows, previous experience with the pile-hammer combination and the confidence level in the analyses.

b) Allowance for underdrive or overdrive: With piles having thickened sections at the seafloor, an extra length of heavy wall material in the vicinity of the seafloor may be used so that the pile will not be overstressed at this point if the design penetration is not reached. The amount of underdrive or overdrive allowance provided in the design will depend on the degree of uncertainty regarding the penetration that can be obtained.

c) Driving shoe: The purpose of a driving shoe is to assist piles to penetrate through hard layers or to reduce driving resistance, thereby allowing greater penetrations to be achieved than would otherwise be the case. If an internal driving shoe is provided for driving through a hard layer, it shall be checked that the driving shoe does not reduce the end bearing capacity of the soil plug below the value assumed in the design. If an internal driving shoe is used for reducing the internal skin friction during driving in cohesive soils, the effect of the driving shoe shall be taken into account when evaluating the total representative capacity of the pile. External driving shoes are not normally used, as they tend to reduce the skin friction along the length of pile above them.

d) The tip of the pile or the driving shoe should be flat or bevelled towards the inside of the pile. Pile tips and driving shoes with bevels toward the outside of the pile shall not be used when driving through dense and very dense sands as they have been shown to be a contributing factor in observed pile buckling.

8.7.7 Use of hydraulic hammers

Hydraulic hammers are more efficient than steam hammers and the energy transferred to the pile for a given rated energy tends to be greater. They can be used both above and below water for driving battered or vertical piles, through legs or through sleeves and guides, as well as vertical piles alone without lateral restraint. In calculating pile stresses, full account should be taken of wave, current and wind actions, as well as the hammer and section weights both during driving and during hammer stabbing, which can be either above or below water. While for steam hammers the weight of the cage is generally held by a crane, for hydraulic hammers the whole weight of the hammer is borne by the pile.

The energy output is generally varied to maintain a low blow count. Therefore, blow counts do not give a direct guide to soil stratification and resistance. Since the ram is encased, hammer performance cannot be judged visually. The hammer's performance, including ram impact velocity, stroke, pressure of accelerating medium and blow rate, should therefore be measured. Reliable instrumentation of some piles may also be considered to verify the energy transferred to the pile to aid interpretation of soil stratification and to limit pile stresses.

Monitoring of underwater driving requires that easily identified, unambiguous datum points be used, together with robust television cameras or remotely operated vehicles (ROV) capable of maintaining station. Alternatively, for shallow water sites, hammer casing extensions or followers may be used so that blow counts can be monitored above water.

Because no cushion block is used, there is no change in characteristics between ram and anvil as driving progresses and no requirement for cushion changes.

In selecting hydraulic hammers for deep water applications, account should be taken of possible decrease in driving efficiency due to increased friction between the ram and its surrounding air. Sufficient air should be supplied to the hammer so that water ingress is prevented. Water in the pile should be able to escape freely.

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8.7.8 Drilled and grouted piles

There are two types of drilled and grouted piles.

a) Single-stage piles: For the single-stage drilled and grouted pile an oversized hole is drilled to the required penetration, a pile is lowered into the hole and the annulus between the pile and the soil is grouted. This type of pile can be installed only in soils which will hold an open hole to the seafloor.

b) Two-stage piles: The two-stage drilled and grouted pile consists of two concentrically placed piles grouted to become a composite section. A pile is driven to a penetration which has been determined to be achievable with the available equipment and below which an open hole can be maintained. This outer pile becomes the casing for the next operation, which is to drill through it to the required penetration for the inner or 'insert' pile. The insert pile is then lowered into the drilled hole, and the annuli between the insert pile and the soil and between the two piles are grouted. The diameter of the drilled hole should be at least 150 mm (6 in) larger than the insert pile diameter.

The hole for drilled and grouted piles can be drilled with or without drilling mud to facilitate maintaining an open hole. Drilling mud can be detrimental to the surface of some soils. If used, mud should be flushed with circulating water upon completion of drilling, provided the hole will remain open. Reverse circulation should normally be used to maintain sufficient flow for removal of cuttings. Drilling operations should maintain proper hole alignment and minimize the possibility of hole collapse.

Centralizers should be attached to the pile to provide a uniform annulus between the insert pile and the hole. A grouting shoe may be installed near the bottom of the pile to permit grouting of the annulus without grouting inside the pile. If a grouting shoe is used, the pile should be tied down to prevent floatation in the grout. The time before grouting the hole should be minimized in soils which can be affected by exposure to sea water. The quality of the grout should be tested at intervals during the grouting of each pile. Means should be provided for determining that the annulus is filled. Holes for closely positioned piles should not be open at the same time, unless there is verification that this will not be detrimental to pile capacity and that grout will not migrate during placement to an adjacent hole.

8.7.9 Grouting pile-to-sleeve connections

The grout-to-steel bond in the connection between pile and sleeve shall be checked in accordance with ISO 19902 addressing grout connections.

8.7.10 Pile installation data

Throughout the driving of piles, comprehensive driving and associated data should be recorded and reviewed for conformance with the installation plan. If significant deviations are observed, corrective measures can be necessary. The recorded data can include the following:

— structure and pile identification, water depth and reference elevation of readings of pile markings for pile tip penetration;

— relevant information on pile stabbing;

— penetration of the pile under its own weight or under the weight of a new add-on;

— additional penetration of the pile under the weight of the hammer;

— data on followers used, where applicable;

— blow counts throughout driving, with hammer identification and hammer blow rate (blows per minute) after every few metres of penetration;

— cumulative number of blows at relevant penetrations;

— driving energy observations and hammer monitoring data, if available;

— pile instrumentation data, if available;

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— date and time of starts and stops in driving, including set-up time;

— elapsed time for driving each section, with actual length of pile sections and cut-offs;

— unusual behaviour of the hammer or the pile during driving;

— elevations of soil plug and internal water surface after driving;

— pertinent data of a similar nature covering drilling, grouting or concreting of grouted piles.

8.7.11 Installation of conductors and shallow well drilling

The planning and execution of conductor installation and shallow well drilling should recognize the potential for disturbance to foundation soils and the consequent risk of a reduction in stability of the fixed structure or of adjacent conductors.

During drilling operations, soil disturbance can result from hydraulic fracture, from wash-out or from encountering shallow gas pockets. Hydraulic fracture occurs where drilling fluid pressure is too high and fluid is lost into the formation, possibly softening the surrounding soil. Wash-out (uncontrolled enlargement of the drilled hole) generally occurs in granular soils and can, in part, be induced by high drilling fluid circulation rates. Wash-out leads to stress relief in the surrounding soils. These incidents can be accompanied by loss of circulation of drilling fluids, by return of these fluids to the seafloor other than through the conductor, or by the creation of seafloor craters. In addition, prolonged drilling activities without casing can generate an upwards seepage flow that reduces effective overburden pressure resulting in reduced axial friction capacity of conductors.

If piles are installed within the zone of influence of soil disturbance, reduction in axial or lateral capacity and foundation stiffness can occur. Similarly, the stability of shallow foundations can be reduced and settlements increased. These detrimental effects can occur whether the drilling takes place after installation of the structure or before (e.g. for a pre-installed template or for an exploration well). Conductors installed by drilling or drill-drive techniques can have more detrimental effects than conductors installed by driving alone.

The following recommendations should be evaluated for conductor installation and shallow well drilling.

— The conductor setting depth should be selected taking due account of hydraulic fracture pressure profiles. The depth should preferably be chosen at a cohesive stratum which is a sufficient distance from the proposed pile tip penetration to minimize the risk of disturbance of foundation soils.

— In conductor or shallow well drilling operations, fluid pressures should be kept within the calculated hydraulic fracture pressure profile. Flow rates should be controlled to minimize wash-out, particularly in granular soils.

Records of conductor installation and shallow well drilling should be available. The implications for foundation soils of any incidents, of excessive loss of circulation, of return of drilling fluids to the seafloor other than through the conductor, or of creation of seafloor craters shall be assessed. The cuttings from the well drilling operation, if allowed to accumulate on the seafloor, shall be taken into account in the foundation design (including settlement), installation procedure and structure removal.

The skin friction capacity of conductors installed in cohesive soils by jetting is covered in 10.5.2.

9 Assessment of pile capacity for existing structures

9.1 General

Assessment of pile capacity for existing structures shall be in accordance with ISO 19901-9 which describes the structural integrity management (SIM) activities required to demonstrate the fitness-for-service of existing fixed steel offshore structures throughout the intended design service life.

In the context of the assessment of the foundation system of an existing structure, this involves a review of available geotechnical and foundation data, evaluation of how the original geotechnical design was

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established and potentially differs from current geotechnical practice, any changes in the use or loading of the system, the effects of drilling and production operations, and the effects of time since the structure installation on the performance of the foundation system.

9.2 Geotechnical and foundation data

9.2.1 Geotechnical data

The original geotechnical site investigation used to establish the foundation response is not always based on current practice. The validity of the existing site and soil investigation data shall be assessed. Older site investigation data may be adjusted to reflect differences between older and more recent site investigation data according to ISO 19901-8 and ISO 19901-10 [139], [271].

More recent soil investigation data or geophysical site investigation data can be available from the project site or from nearby locations in the same geological environment. The distance within which non-site-specific data are relevant to the project site depends on the geological environment. Applicability of nearby data where geotechnical stratigraphy and properties are spatially variable shall be assessed based on the understanding of the geological environment.

The feasibility and value of acquiring modern geophysical and geotechnical data close to an existing infrastructure shall be assessed.

9.2.2 Design data

The methods, decisions, geotechnical data and analyses used to demonstrate that the foundations are fit-for-service shall be documented.

Original foundation design data, including the selection of geotechnical parameters, loading criteria and design methods shall be assessed against Clause 8.

If foundation data or geotechnical data are missing or inaccurate, this additional uncertainty shall be addressed in the choice of representative values and methodologies.

9.2.3 Installation data

As-built foundation installation data including pile driving records and dynamic pile monitoring data shall be used, where available, to back-analyse geotechnical stratigraphy and parameters.

Available installation records shall be used to verify as-built data, such as pile lengths, depths to wall thickness changes, and steel material grades which can differ from those given on design drawings, particularly if the piles were driven to refusal and are short of the original design penetration.

9.2.4 Condition data

Foundation condition data shall include as available:

a) historical condition data relating to changes made to foundations;

b) present condition data relating to the surveyed condition of foundations.

Measurements of scour around existing foundations and measurement-based predictions of future scour may be used in preference to the assumptions made in the original design.

Information on the assessment of foundation capacity based on the historical performance of the pile foundation system is provided in 9.3.

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9.2.5 Operational data

Operational data should include, as available, information on:

a) changes in platform actions;

b) exposure to metocean conditions, earthquakes, sediment transport and other environmental events;

c) exposure to accidental events, such as snag actions;

d) envelopes used in operating equipment.

9.3 Evaluation

Quantitative evaluation of existing data shall be undertaken to

a) confirm structural integrity, mitigation strategies, and established risk levels to achieve the required performance level, or

b) to identify that mitigation measures are required.

Such quantitative evaluation shall consider:

— geotechnical and foundation data described in 9.2;

— foundation capacity and response data based on current practice;

— criteria data such as updated knowledge of geohazards at the project site (e.g. earthquake or seabed instability) that can result in a degradation of the foundation soil;

— incident / accident data (e.g. extreme or abnormal seismic event);

— learnings from similar foundation systems or similar geotechnical conditions.

9.4 Assessment

9.4.1 General

ISO 19901-9:2019, 12.3 describes the different methods that can be used for the assessment of existing structures and the foundation capacity. The geotechnical aspects of a pushover response of an offshore platform supported by pile foundations (ultimate strength method) are described in 9.4.2, with reference to ISO 19901-9:2019, 12.3.4.3.

9.4.2 Pushover response of pile foundation systems

The ultimate overall system (pushover) response and capacity of the pile foundation system is characterized by the failure mechanism. For structures in shallow waters dominated by horizontal actions, the failure mechanism is typically shear, and the system capacity is governed by the lateral response of the piles and well conductors. For structures dominated by overturning moments, the failure mechanism is typically overturning, and the system capacity is primarily governed by the axial response of the piles with some contribution from the moment capacities of the piles and well conductors.

With reference to ISO 19901-9, the pile foundation and conductor system shall be treated as follows in pushover analyses.

a) Model the connection between the pile foundations and the structure in the analyses, with a focus on the rotational stiffness of the system.

b) Include jacket leg stubs that extend below the seafloor in the structural analyses. These leg stubs contribute to the lateral capacity of the piles.

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c) Include the well conductors in the structural model of the platform as they can contribute both to the shear resistance and the overturning resistance of the foundation system [139]. Well conductors are modelled in the same way as pile foundations below the seafloor, considering compatibility between forces and displacements between the conductors and the soil and between the conductors and the platform. Conductors shall be modelled to reflect the actual conditions of their guide systems and connections to the jacket.

d) Include the effects of cyclic actions. In Reference [351], it is demonstrated that the use of the cyclic *p-y* curves described in 8.5.2.3 in pushover analyses gives a good hindcast of observed platform performance (i.e. structural damage and pile head displacement at the seafloor) during extreme events in the Gulf of Mexico.

e) Check the sensitivity of the foundation system capacity to the lateral and axial capacities of the piles, where the pile foundation system governs the capacity of the platform. If the foundation system capacity is more sensitive to the lateral soil resistance, the failure mode for the pile system is dominated by shear. If the foundation system capacity is more sensitive to the axial soil resistance, the failure mode is dominated by overturning. In such cases, geotechnical factors such as the stratigraphy and the soil properties will govern the response.

f) Check the sensitivity of the pushover capacity for the pile foundation system to the steel yield strength for piles and well conductors. The rated yield strength can underestimate the most likely or expected value [67]. Use unbiased (rather than low) estimates of input values in calibrating historical performance and in predicting future performance.

g) Account for the fact that the axial load-displacement response of the pile foundation system as represented by load-transfer (*t-z* and *Q-z*) curves can result in an ultimate capacity which is lower than the peak ultimate capacity given in design according to the methods given in 8.1. This is due to the shape of the load-transfer curves and flexibility of the pile. Further guidance is provided in 8.1 and A.8.1.3.2.4 and A.8.4.1.

Survival of a pile foundation system during previous extreme action events shall not be directly used as evidence for increased in situ strength of the soil compared to the original design. Any approach accounting for previous loading events shall address a number of factors of relevance to the entire platform system, as discussed in Reference [139].

9.5 Time-dependent effects on pile foundations

In silica sand, significant increases in axial pile capacity with time have been measured over periods of days up to around one year that are unrelated to pore pressure dissipation (e.g. References [72] and [190]). The mechanisms for this increase in pile capacity, commonly known as “ageing”, are still unproven. Ageing is the effect of age (i.e. time after installation) on the axial capacity of piles. Ageing appears to be related to corrosion and other effects that can raise radial effective stresses and produce a stronger dilation mechanism and higher steel pile-soil interface angles of friction. Loading piles to failure disrupts the ageing process and piles that are loaded to failure show a loss of capacity if re-tested after unloading.

Pile driving in clay normally results in the development of excess pore pressures that produce a low initial axial pile capacity immediately after driving that increases with time as the pore pressures dissipate (see A.8.1.3.2.5). This reconsolidation-related change in axial pile capacity can take months or years, depending on the consolidation characteristics of the clay and the diameter of the pile. However, it has been shown that capacity ageing effects also develop in clay, running in parallel and in addition to the reconsolidation process [190].

Although ageing has been measured on onshore small to medium-scale piles (e.g. the EURIPIDES research piles in very dense silica sands in the Netherlands [194], reliance on ageing effects on large diameter offshore piles experiencing high degrees of long-term cyclic loading without supporting evidence can be inappropriate and the use of the above results shall be validated. The benefit of time-dependent effects on axial capacity may be taken where it can be demonstrated to be available. However, no ageing shall be used when no evidence of ageing effects is available (e.g. for piles driven in carbonate soils). Instrumented re-strike tests can provide evidence of field ageing trends

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10 Geotechnical design input to subsea structures, risers and flowlines

10.1 General

This clause outlines criteria and recommendations pertinent to geotechnical aspect of design of subsea structure foundations, risers and flowlines. It covers methods that are not addressed in other international standards (e.g. ISO and API) and DNV guidelines. Unless noted, the recommendations provided in this clause are applicable to all seabed soils and conditions. Local experience and practices, where exist, may be used in lieu of the methods presented here.

10.2 Geotechnical investigation

A ground model is required for deriving the input to soil-structure interaction. The ground model may be developed by integrating data from geotechnical, geophysical and geological site investigations. The geotechnical site investigation may include in-situ testing and soil sampling complemented by laboratory testing. The depth of the ground model is determined based on the geometry and design requirements of the subsea structure. Details about equipment and procedures for marine soil investigations are provided in ISO 19901-8. In addition to index and classification tests (i.e. bulk density, Atterberg limits, moisture content, grain size distribution, specific gravity, and carbonate content), undisturbed and remoulded shear strength shall be measured. An understanding of in situ OCR and expected dilatant or contractant behaviour of the soil when sheared can prove useful for the design of flowlines. Depending on the intended application, pH, thermal conductivity and electrical resistivity tests can also be performed to assess the insulating and corrosive properties of the soil.

10.3 Foundations for subsea production structures

Foundation configurations that can be utilized include mudmats, driven piles, pushed pin piles, suction piles, jetted conductors or combinations of these.

Mudmats used to support subsea production structures shall have skirts. The design and installation of shallow and intermediate foundations with skirts is addressed in Clause 7 and Clause A.7. The design and installation of piled foundations is addressed in Clause 8 and Clause A.8.

Contingency methods should be established for situations where the foundation fails to penetrate the seabed to the target penetration. A minimum of one alternate installation location should be selected. Where deep foundations are used, the alternate location should be a minimum of:

a) two diameters, centre-to-centre, from primary location for the case of driven piles and jetted conductors; and

b) three diameters, centre-to-centre, for extracted suction caissons while demonstrating this distance allows the suction seal to be maintained during re-installation. However, if no evidence of disturbance in the soil plug is observed, this distance can then be reduced to one and a half diameters, centre-to-centre.

Where shallow foundations are used, the design should demonstrate alternate location is not impacted by mudmat extraction.

10.4 Steel catenary risers

10.4.1 General

A steel catenary riser (SCR) refers to portion a subsea pipeline that rises from seafloor to a deepwater floating production vessel. The zone at which the SCR contacts the seabed during cyclic motions is referred to as the touchdown zone. Design and assessment of SCRs involve ultimate limit state and fatigue limit state analyses both of which, require soil response modelling as input.

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10.4.2 Seabed characterisation

Geotechnical site investigation shall be carried out to within the touchdown zone for developing soil response for input to SCR-soil interaction analysis. The geotechnical site investigation:

a) shall be conducted in accordance with the requirements in ISO 19901-8;

b) shall include box cores;

c) should include piston cores or gravity cores;

d) should collect the following minimum soil sample core lengths:

   1) 0,5 m for box cores;

   2) 3 m for piston cores or gravity cores;

e) shall include monotonic and cyclic mini T-bar or full-flow penetrometers test performed offshore in the box cores;

f) should include minivane test performed offshore in the box core;

g) shall include minivane test performed on piston or gravity cores, if collected.

10.4.3 Design for ultimate limit state

An ultimate limit state can arise under extreme environmental events that cause out-of-plane motion, particularly when the riser becomes embedded or lies within a trench, thus giving rise to high lateral soil resistance and locally high curvature of the riser. During out-of-plane motion, the riser will encounter resistance from the sides of any trench or soil berms formed on either side of the pipe. Specialist geotechnical advice should be sought to quantify the lateral soil resistance, which usually exceeds the normal frictional resistance for pipelines lying on the seafloor [4].

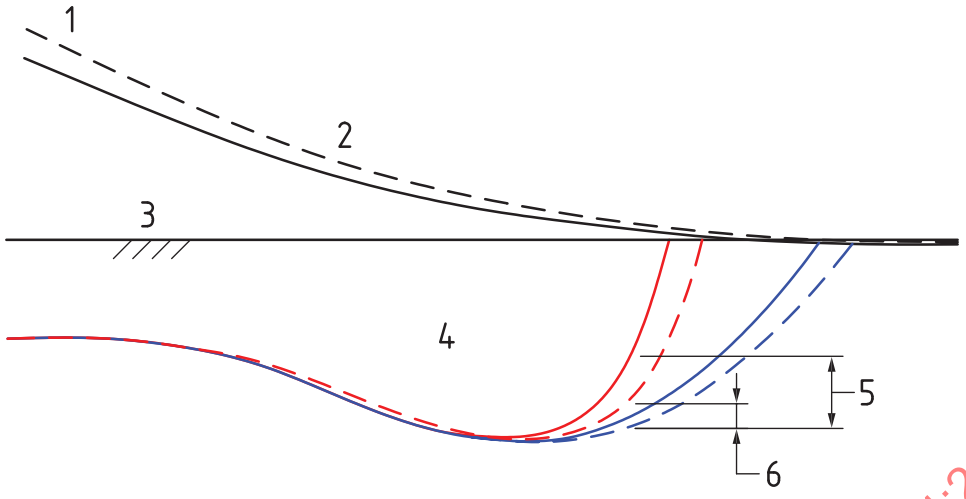
10.4.4 Design for fatigue limit state

10.4.4.1 General

The stress ranges used in the fatigue analysis of SCRs are calculated from the changes in riser stress caused by first- and second-order motions (see Figure 13). Within the touchdown zone these motions can be simplified to moving the touchdown point (TDP) in-line with the riser and assessing the resulting changes in bending moment. Fatigue damage in the riser depends particularly on the soil stiffness within the touchdown zone. Riser cyclic action remoulds the seabed and degrades the stiffness.

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Key

1	simulated riser motion	4	stress envelopes along riser length near the TDP
2	riser	5	stress range, Δσ – stiff soil
3	seafloor	6	stress range, Δσ – soft soil

Figure 13 — Example stress changes for SCR fatigue calculations

Three soil modelling approaches are available for developing soil response to fatigue assessment of SCRs in clay seabed:

a) an elastic soil spring along the entire touchdown region;

b) a non-linear elastic spring based on the soil response in a 'fully degraded' state;

c) a non-linear spring that considers loading, unloading, reloading cycles, penetration into the seafloor and variable shear strengths with depth.

The approaches increase in complexity with their order and are only pertinent to soil response in the vertical plane of the riser motion in the water column. The first approach is the least complex and involves representing the seabed response to cyclic loading through a single elastic soil spring. The second approach is based on the assumption of fully remoulded soil response but with a stiffness that varies as a function of riser penetration.[29] The third approach, although the most complex. It tracks the soil response from the first cycle of SCR motion. See A.10.4 for guidance on use of the approaches for SCR-soil interaction analysis. For the second approach, model parameters for the Gulf of Mexico and offshore Angola deepwater seabed conditions are provided. [285]

10.4.4.2 Selection of s_u below SCR

In certain regions of the world, a crust of higher strength soil exists in the upper 0,5 to 1 m, before the strength profile reverts to a linear trend. The potential for the SCR to punch through the crust, and the consequences for fatigue studies, shall be evaluated. The initial SCR penetration into the seafloor can be estimated from [30]:

$$Q_u = N_c \cdot s_u \cdot D \approx a(z/D)^b \cdot s_u \cdot D \quad (42)$$

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where

Q_u is the limiting penetration resistance per unit length;

D is the pipe diameter;

N_c is the bearing capacity factor between 5 and 6;

s_u is the shear strength at pipe invert;

a and b are model parameters with average values of about 6 and 0,25.

The depth of the maximum expected pipe penetration shall be:

a) calculated using Formula (42);

b) reconciled with field data in similar conditions including soil profile and type of production platform, if available.

The value of s_u to be used in SCR-soil interaction analysis in design cases shall be as follows.

— For sites where the soil shear strength profile does not exhibit a crust, s_u is taken as the soil shear strength at depth of maximum expected pipe penetration.

— For sites where the shear strength profile exhibits a crust:

  — s_u = average shear strength between the depth of maximum expected pipe penetration and depth of 0,5 m below seafloor, if risk of pipe punch-through the crust is low;

  — s_u = average shear strength between the depth of maximum expected pipe penetration and depth of 1 m below seafloor, if risk of pipe punch-through the crust is high.

Potential for punch-through is typically assessed to be low for vessels where heave motion is not expected to be high (e.g. TLP, spar) and high for vessels where heave motion is expected to be high (e.g. platforms with semi-sub or ship shaped hulls).

10.4.4.3 Trenching effects

In design cases in undeveloped site, the SCR-soil analysis model shall:

a) account for the general seafloor slope at the touchdown zone;

b) assume no trench is present at the seafloor.

In assessment of existing SCRs:

— analysis shall account for the general seafloor slope at the touchdown zone.

— shape of the trench profile should be measured in the field.

— SCR-soil analysis model should include the measured shape of the trench profile.

Field surveys have shown that in some cases trenches a few metres deep can develop [54]. [286] The effect of trenches on riser fatigue life and options for investigating such effect are provided in A.10.4.4.3.

10.5 Geotechnical design for jetted conductors and top tension risers

10.5.1 General

The geotechnical design of top tension risers is focused on the conductor which is the uppermost section of the well positioned in the seabed. The conductor section provides lateral support under short term ultimate limit state conditions and longer-term fatigue.

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10.5.2 Jetted conductors

10.5.2.1 General

ISO 3421 outlines the requirements for design, setting depth and installation of offshore well conductors. It addresses determination of diameter, wall thickness and steel grade as well as determination of the setting depth for three installation methods, namely, driving, drilling and cementing and jetting along with the requirements for procedures, documentation and quality control. In the absence of existing practice and already established installation techniques, the approaches outlined in 10.5.2 can be used to estimate short-term (<10 days) and long-term (>10 days) axial bearing capacity of jetted conductors installed in normally consolidated to lightly over-consolidated clays. The method outlined herein was initially developed for the Gulf of Mexico clays and, through basin-specific calibration, its use has been extended to other regions. It can be applied to basins not covered in this subclause after calibration to basin-specific conditions, but shall not be used in sands.

10.5.2.2 Short-term axial bearing capacity

Conductor configuration and jetting bit, tool joint information and bottom hole assembly information shall be determined for design.

Prior to designing a jetted conductor, a feasibility study shall be performed to assess if site conditions are favourable for installing the conductor using jetting. The following data, as available, shall be reviewed to assess likelihood of premature refusal:

a) geotechnical data, including the presence of sand strata and stiff clays near the seafloor;

b) geophysical data, including the presence of erosional unconformities which can imply stiff to very stiff clays at shallow depths;

c) past installation records, if any.

The immediate axial capacity of the conductor is defined at a time equal to 0,01 day (i.e. 14 mins). In the absence of more definitive criteria, the immediate axial capacity shall be estimated using Formula (43).

$$Q_o = WOB_{last} = R \cdot (W_{cond} + W_{WH} + W_{DC} + W_{CADA}) \quad (43)$$

where

Q_o is the jetted conductor axial capacity immediately after jetting (t=0,01 days);

WOB_{last} is the last weight on bit recorded during installation;

R is the WOB utilization ratio;

W_{cond} is the weight of the surface conductor in water;

W_{WH} is the weight of the wellhead housing in water;

W_{DC} is the weight of the drill collars in water;

W_{CADA} is the weight of the drill-ahead tool in water.

The WOB utilization ratio, R, during reciprocation shall be kept less than 1,0 to avoid compression stresses and prevent buckling in the bottom hole assembly (BHA) and running string. An R value of 0,8 should be used.

Set-up of jetted conductors shall be estimated using Formula (44).

$$\Delta\alpha_t = \frac{Q_t - Q_o}{\pi \cdot D \cdot L \cdot s_{uave}} \quad (44)$$

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where

$\Delta\alpha_t$ is the change in average friction factor along the conductor, at time t, due to set-up;

Q_t is the conductor capacity at time = t days < 10 days;

D is the surface conductor diameter;

L is the conductor length below seafloor (i.e. embedded length);

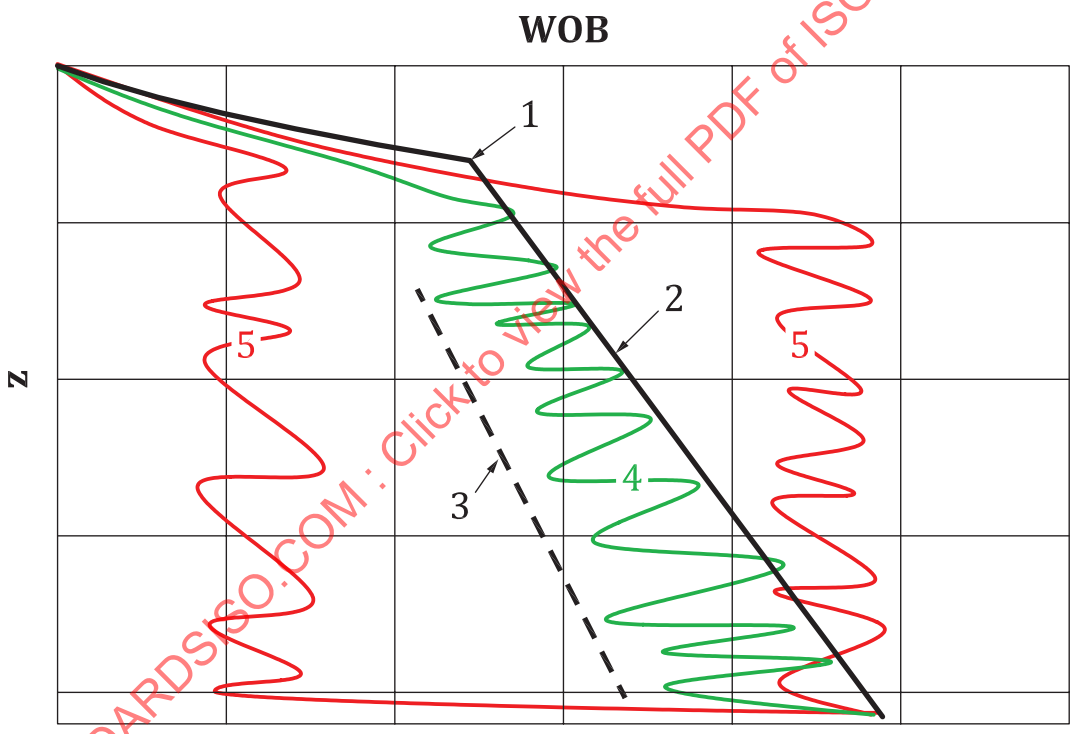
$s_{u_{ave}}$ is the average undrained shear strength over the embedded length of the conductor.

The $\Delta\alpha_t$ relationship with time is developed through regional experience from back-analysis of installed jetted conductors. In the absence of more definitive criteria, Formulae (45) to (47) shall be used provided that:

— the design WOB profile is constructed based on the criteria outlined in Reference [179];

— the drill bit stick-out is between 75 mm and 228 mm (3 in and 9 in) [1];

— the conductor reciprocations during installation are performed such that the WOB remains between the minimum and design lines as shown in Figure 14.



Key

1 end of self-weight penetration
2 design or recommended maximum WOB profile for jetting
3 design or recommended maximum WOB profile for jetting (80 % of maximum design profile)
4 desirable WOB profile
5 undesirable WOB profile
z depth below original seafloor

Figure 14 — Schematic examples of weight-on-bit (WOB) profiles: design or recommended, desirable and undesirable

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For Gulf of Mexico (see Reference [179]), offshore Mauritania, Senegal, and Egypt clays:

$$\Delta\alpha_t = 0{,}055 \cdot [2 + log(t)] \quad (45)$$

For Capsian Sea clays:

$$\Delta\alpha_t = 0{,}035 \cdot [2 + log(t)] \quad (46)$$

For offshore Angola clays and conductor length less than 69 m [118]:

$$\Delta\alpha_t = 0{,}038 \cdot [2 + log(t)] \quad (47)$$

For offshore Angola clays and conductor length between 69 m and 82 m [118]:

$$\Delta\alpha_t = 0{,}029 \cdot [2 + log(t)]$$

The short-term (<10 days) conductor axial bearing capacity at time t shall be estimated using Formula (48).

$$Q_t = WOB_{last} + \Delta\alpha_t \cdot \pi \cdot D \cdot L \cdot s_{u\,ave} \quad (48)$$

Jetted conductors are considered temporary foundations. To account for uncertainty in action and soil resistance, action factors and partial resistance factors shall be applied to achieve a minimum overall factor of safety of 1,3.

10.5.2.3 Long-term axial bearing capacity

Jetted piles are piles installed with the jetting technique used for jetting conductors and can be used as permanent foundations for subsea structures. In the normally consolidated to lightly over-consolidated Gulf of Mexico clays and in absence of site-specific data, the long-term (>10 days) axial capacity of jetted piles shall be estimated using Formula (48) with the set-up relationship given in Formula (45) [365]. Action factors and partial resistance factors equal to those for driven piles shall be used in design, which is often controlled by operating, not extreme, conditions.

10.5.3 Soil-structure interaction for well integrity assessment

The requirements and guidance provided in 10.5.4 and 10.5.5 are for developing soil input parameters for integrity assessment of top tensioned riser (TTR) systems. The requirements and recommendations provided in 10.5.4 and 10.5.5 for well strength and fatigue assessment are exclusive of any factors of safety or partial factors to loading actions and resistances. Factors of safety may be selected through industry recommended practices (see References [24], [25] and [101]) or chosen by operators based on regional experience or case specific basis.

10.5.4 Geotechnical input to well strength assessment

10.5.4.1 Clays

The lateral capacity of an element of the well conductor in normally consolidated to lightly over-consolidated clays with s_u less than 100 kPa for a well strength analysis shall be assessed using the approach in the main text for the monotonic lateral response of piles in 8.5.2.2.

10.5.4.2 Sands

The lateral capacity of an element of the well conductor in sand for a well strength analysis can be assessed using the approach for the monotonic lateral response of piles in sand in 8.5.3.

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10.5.5 Geotechnical input to well fatigue assessment

10.5.5.1 General

The guidance provided in this subclause does not apply to the following unconventional soils and seabed conditions:

a) calcareous and carbonate soils;

b) structured soils with sensitivity greater than 8,0 or slicken-sided;

c) hard glacial till seabed where gravel, cobbles, boulders or hardpan can exist;

d) seabed soils with shallow gas or gas hydrate;

e) soils disturbed by operating activities;

f) soils contaminated by cement and drill cuttings.

Two methods are presented for fatigue assessment of wells in clays: the spring-only method and the spring-dashpot method. The spring-dashpot method is more appropriate where the assessment requires estimation of the dynamic behaviour of the TTR system. The spring-dashpot method accounts for the energy dissipated in the foundation soil during each load-unload loop, termed as hysteretic damping. Experiments and field observations have demonstrated that hysteresis can be important for fatigue assessment of TTR systems installed in clay seabeds. More details are provided in A.10.5.5.

Development of p_{fa}-y_{fa} data shall be based on the best estimate soil strength properties, if site-specific soil properties are available. In the absence of site-specific geotechnical information, engineering judgement can be used to derive the best estimate soil parameters. Sensitivity analysis should be performed with p_{fa}-y_{fa} data derived using upper estimate and lower estimate soil properties, if site-specific soil properties are available. Otherwise, engineering judgement may be used to derive the upper and lower estimates of the soil properties. Laboratory tests can be performed on site-specific samples to directly obtain p_{fa}-y_{fa}. An example of laboratory testing equipment and procedures to obtain p_{fa}-y_{fa} data for well fatigue analysis is presented in Reference [358].

For developing p_{fa}-y_{fa} springs, the following general provisions are given:

— parameters in calculations shall be determined for the pfa-yfa spring depth;

— first soil spring should be at seafloor;

— spacing of p_{fa}-y_{fa} curves along the conductor shall be no greater than:

  — 0,5 m in the top 25 m below seafloor;

  — 1,0 m below 25 m depth below seafloor;

— analyses shall include the following model conditions, unless field observations indicate otherwise:

  — no seafloor scour or erosion;

  — no gapping between the conductor and the seabed;

  — no drill cuttings and cement at seafloor;

— the ground model and soil parameters shall extend to at least 45 m below seafloor for the purpose of developing p_{fa}-y_{fa} curves for well fatigue;

— p_{fa}-y_{fa} curve formulations shall only be used for fatigue analysis where rig and riser are in a no-offset position (see A.10.5.4 and A.10.5.5 for discussions on offset conditions).

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10.5.5.2 Clays

10.5.5.2.1 Spring-only method

The lateral p_{fa}-y_{fa} response of a soil element for well fatigue assessment in clays shall be assessed using the approach outlined in 8.5.2.4.

10.5.5.2.2 Spring-dashpot method

Soil damping effects can be approximated by introducing equivalent viscous damping with energy dissipation equal to that dissipated by hysteresis. The viscous damping is modelled by a dashpot having a viscous coefficient, c, expressed by:

$$c = \frac{2^{K_{Sec._SS}} . \xi}{\omega} \tag{49}$$

where

$K_{Sec.SS}$ is the soil secant stiffness at the steady-state condition;

ξ is the soil damping ratio;

ω is the motion angular frequency obtained from the modal wave frequency, in riser analysis.

Estimation of the viscous coefficient uses a first approximation of displacements along the well. In the absence of more definitive criteria, the following steps shall be used:

a) perform analysis using spring-only approach as outlined in 8.5.2.4 and obtain displacement amplitude time-histories 4,0 m above seafloor;

b) determine ξ profile for each sea-state motion based on the profiles provided in Figure 15 for maximum one-way displacement amplitudes taken at 4,0 m above seafloor; interpolation can be used where a displacement amplitude (half of peak-to-peak) falls between the provided profiles;

c) determine $K_{Sec._SS}$ for each sea-state motion using Figure 16 with the maximum value not exceeding 700 s_u (with s_u taken at the spring-dashpot depth);

d) calculate c values at each spring location using Formula (49);

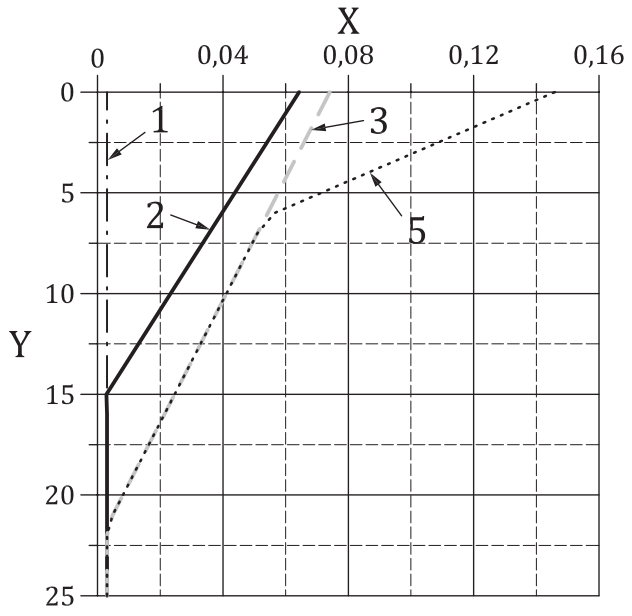
e) re-run fatigue analysis with a dashpot viscous coefficient calculated from Formula 34 and p_{fa}-y_{fa} curves calculated from 8.5.2.4 and Formula (34), but with the following A_s and B_s model parameters:

   1) if $s_u < 40$ kPa, A_s = 0,335 and B_s = -0,03;

   2) if $s_u \geq 40$ kPa, A_s = 0,27 and B_s = -0,175.

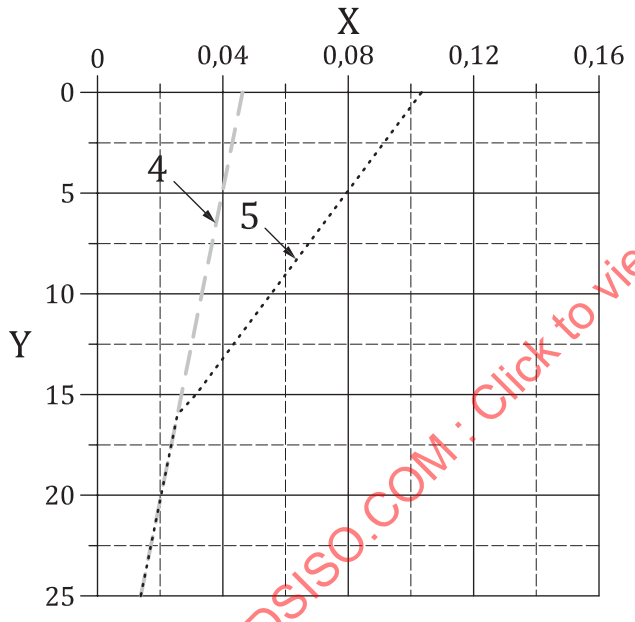
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a) Normally consolidated to lightly over-consolidated clays

Depth BML	Maximum y amplitude			
m	<0,025D	0,025D	0,05D	0,01D
0,0	0,003	0,064	0,074	0,146
7,0	0,003	0,036	0,051	0,051
15,0	0,003	0,003	0,025	0,025
22,0	0,003	0,003	0,003	0,003
25,0	0,003	0,003	0,003	0,003



b) Over consolidated clays

Depth BML	Maximum y amplitude	
m	≤0,05D	0,1D
0,0	0,046	0,103
16,0	0,026	0,026
25,0	0,014	0,014

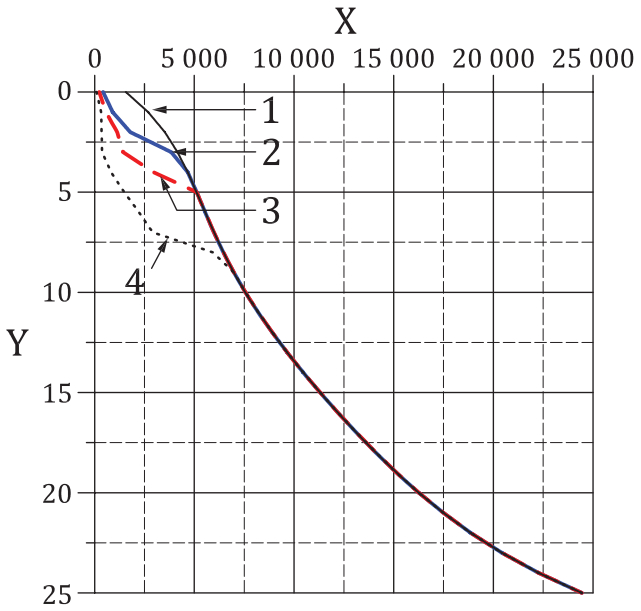
Key

X soil damping ratio, ξ
Y depth below original seafloor, z (m)
1 maximum y amplitude < 0,025D
2 maximum y amplitude = 0,025D
3 maximum y amplitude = 0,05D
4 maximum y amplitude ≤ 0,05D
5 maximum y amplitude = 0,1D
D conductor outside diameter

Figure 15 — Clay soil damping ratio profiles for maximum one-way (half of peak-to-peak) well displacement amplitudes taken at 4,0 m above seafloor [359],[360]

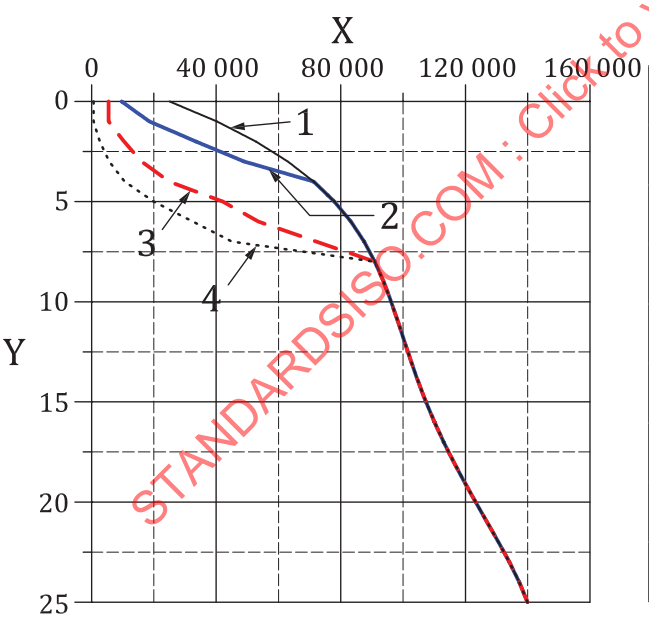
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a) Normally consolidated to lightly over-consolidated clays

$K_{Sec,_SS}$ value for maximum y amplitudes					
Depth (m)	**0,025*D***	**Depth (m)**	**0,05*D***	**Depth (m)**	**0,1*D***
0,0	421	0,0	224	0,0	95
1,0	905	1,0	561	1,0	326
2,0	1 786	2,0	1 123	2,0	350
3,0	3 844	3,0	1 421	3,0	369
4,0	4 670	4,0	2 943	4,0	823
5,0	5 119	5,0	5 119	5,0	1 480
>5,0	700 s_u	6,0	5 546	6,0	2 216
		>6,0	700 s_u	7,0	2 908
				8,0	5 917
				9,0	6 982
				>9,0	700 s_u



b) Over-consolidated clays

$K_{Sec,_SS}$ value for maximum y amplitudes					
Depth (m)	**0,025*D***	**Depth (m)**	**0,05*D***	**Depth (m)**	**0,1*D***
0,0	9 660	0,0	5 457	0,0	673
1,0	18 541	1,0	5 457	1,0	673
2,0	33 553	2,0	10 454	2,0	2 676
3,0	49 060	3,0	16 078	3,0	5 673
4,0	71 305	4,0	24 917	4,0	10 625
> 4,0	700 s_u	5,0	42 117	5,0	19 916
		6,0	53 642	6,0	32 515
		7,0	72 283	7,0	45 115
		8,0	90 925	8,0	90 925
		>8,0	700 s_u	>8,0	700 s_u

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Key

X	steady-state secant stiffness, $K_{Sec._SS}$ (kN/m/m)	3	maximum y amplitude = 0,05D
Y	depth below original seafloor, z (m)	4	maximum y amplitude = 0,1D
1	700 s_u y = 0,0D	D	conductor outside diameter
2	maximum y amplitude = 0,025D		

Figure 16 — Clay $K_{Sec._SS}$ profiles for maximum one-way (half of peak-to-peak) well displacement amplitudes taken at 4,0 m above seafloor [359] [360]

10.5.5.3 Sands

The lateral capacity of an element of the well conductor in sand for a well fatigue analysis can be assessed using the approach in the main text for the lateral response of piles in sand (see 8.5.3).

10.5.6 Geotechnical considerations in conductor driving analysis

Recommendations for the installation of driven foundation piles are outlined in ISO 19902 and in 8.7. The same procedures apply to conductor driving analyses. Additional guidance on the following subjects pertaining to conductor pipe driving is provided in A.10.5.6:

a) conductor pipe installation;

b) remedial measure when encountering premature refusal during conductor driving;

c) stress verification during hammer placement and driving;

d) fatigue damage to conductor pipe from driving;

e) conductor pipe tip damage and bucking; and

f) use of inclined shoes to drive conductors.

10.6 Foundation design for riser towers

10.6.1 General

The riser tower concept consists of a free-standing riser assembly, incorporating several risers in a bundle configuration, tensioned from the top by a buoyancy tank, and anchored to the seabed. The tower is generally connected to the surface vessel or platform by flexible jumpers.

A riser tower supports axial tension generated by buoyancy and by cyclic wave action and part of the tension can act permanently during the life of the structure.

10.6.2 Foundation options

Foundation options include gravity base, suction piles and driven piles, or a combination of these.

The vertical uplift resistance consists of the following three components:

a) submerged foundation weight;

b) external skin friction;

c) for suction piles, reverse end bearing at the bottom of the pile.

Permanent, i.e. sustained, uplift action can be counter-acted by a ballast weight.

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10.6.3 Loading actions and safety factors

10.6.3.1 Loading actions

Design loading actions shall be evaluated for the following conditions:

a) foundation installation and retrieval;

b) operating conditions;

c) extreme conditions.

10.6.3.2 Recommended safety factors

Design philosophy shall follow ISO 19900 and selection of safety factors. Additional guidance is provided in A.10.6.3.2.

10.6.4 Design challenges

The following general principles apply in assessing the stability of riser tower foundations.

a) The shear strength used in the analysis shall account for effects of creep and potential loss in reverse end bearing from drainage under sustained or long duration actions and cyclic degradation. The potential changes in effective stresses and shear strength due to potential drainage can be studied by finite element analyses.

b) Displacement and deformation during the life of the foundation shall be addressed in design. Where displacement and deformation govern the design, finite element analysis methods may be used (see 7.6 and A.11.5). The displacement analysis shall include contributions from undrained shear strains due to application of the sustained actions, undrained creep during the sustained action, and permanent and cyclic components from the wave loading and currents. Displacements due to shear strain, volumetric strains and flow of water through the soil due to potential drainage during the sustained loading period shall also be addressed.

c) Foundation susceptibility to erosion and scour shall be assessed by means of analysis and/or laboratory testing. Preventative measures shall be implemented, if the seabed foundation is deemed susceptible to scour.

d) Where removal is anticipated, an analysis shall be made of the actions generated during removal so that that removal can be accomplished with the means available.

10.7 Offshore pipelines and flowlines

10.7.1 Geotechnical pipe-soil interaction (PSI) analysis

10.7.1.1 General

A phased approach can be taken to determine geotechnical input to pipe-soil interaction (PSI) analysis for design of new and assessment of existing pipelines and flowlines. The phases can comprise: desk study, preliminary and detailed. The general requirements for provision of geotechnical input to pipe-soil interaction (PSI) analysis are as follows.

a) The ground model shall be developed for PSI analysis at the first step in PSI analysis.

b) Development of the ground model shall include:

   1) review of existing local and regional knowledge relating to pipe embedment and PSI behaviour;

   2) terrain unit mapping to establish soil provinces;

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3) one or more corridors that envelop all potential pipeline routes;

4) best, lower and upper estimates the in situ soil parameters as the first step in a PSI analysis; these parameters shall:

i) include the strength (intact and remoulded), submerged unit weight and pipe-soil interface strength (drained and undrained), coefficient of consolidation;

ii) be defined from the seafloor to the depth of influence of the pipeline;

iii) encompass the uncertainty in the data and the variability along the route.

c) The ground model should be updated during the preliminary and detailed stages.

d) PSI analysis shall address the as-laid or as-installed pipeline conditions, and any subsequent changes in pipeline and seabed conditions during the operating life, and the resulting pipe-soil force–displacement responses in the axial and lateral directions.

e) Best, lower and upper estimates of the pipe–soil interaction forces shall be derived.

f) The geotechnical PSI analysis shall estimate the relevant drainage conditions for each action, including the lay process and subsequent pipeline loading events or movements, in order that the relevant soil responses (drained, undrained, intermediate or multiple responses) are used in the PSI analysis.

g) For axial pipe-soil resistance, the geotechnical PSI analysis shall be based on estimates of the drained and undrained pipe-soil interface strength, allowing for the roughness of the pipe coating, and supplemented by an adjustment for wedging around the curved pipe surface.

h) Where seabed has a gradient, cyclic loading can lead to a pipeline to move down-slope with each successive cycle. Soil parameters for evaluation of cyclic PSI on sloping seabed gradient shall also be developed.

i) If the seabed is potentially mobile (e.g. scour, liquefaction or sediment deposition), the geotechnical PSI analysis shall address through-life changes for the pipeline conditions.

10.7.1.2 Buried pipelines and flowlines

For buried pipelines, the geotechnical PSI analysis shall be in accordance with DNVGL-RP-F110.

10.7.1.3 Unburied pipelines and flowlines

For unburied pipelines, the geotechnical PSI analysis shall be in accordance with DNVGL-RP-F114.

10.7.1.4 Seabed and seafloor characterisation

During the preliminary and/or detailed stage:

— The ground model in the corridors in the top portion of the seafloor within the zone of PSI shall calibrate the geophysical data with the geotechnical data.

— If site investigation is used:

  — It shall be carried out in accordance with ISO 19901-8 requirements.

  — It shall include soil sampling.

  — It shall include in-situ tests or shallow penetrometers.

10.7.2 Submarine slides and density flows: simulation and pipeline impact analysis

In regions where submarine slides and density flows (both referred to as ‘slides’) are anticipated to cross pipeline routes, the risks to the pipeline(s) shall be evaluated.

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11 Design of anchors for floating structures

Geotechnical considerations for the design of anchors for stationkeeping systems for floating structures are given in Clause A.11.

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Annex A
(informative)

Additional information and guidance

NOTE The clauses in this annex provide additional information and guidance. The title of each subclause corresponds with the equivalent (sub)clause in the body of this document.

A.1 Guidance on scope

There is a large body of technical literature on offshore geoscience studies and offshore geotechnical engineering design (see Bibliography). There are also regular conferences on these topics, including:

a) the Proceedings of the International Symposia on Frontiers in Offshore Geotechnics (ISFOG);

b) the Proceedings of the Society of Underwater Technology (SUT) conference on Offshore Site Investigation and Geotechnics (OSIG);

c) the Proceedings of the Offshore Technology Conference (OTC);

d) the Proceedings of the International Symposia on Offshore and Polar Engineering (ISOPE), and on Offshore Mechanics and Arctic Engineering (OMAE).

General guidance on the application of soil mechanics theory to foundation design can be found in various undergraduate and post-graduate text books. Text books covering specific application of soil mechanics to offshore geotechnical design are uncommon [273] [96].

A.2 Guidance on normative references

No additional guidance is offered.

A.3 Guidance on terms and definitions

No additional guidance is offered.

A.4 Guidance on symbols and abbreviated terms

No additional guidance is offered.

A.5 Guidance on general requirements

A.5.1 General

No additional guidance is offered.

A.5.2 Design cases and partial factors

No additional guidance is offered.

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A.5.3 Representative and design values of geotechnical parameters

A.5.3.1 Guidelines

The representative value for a specific soil property can be a single value, a single value per soil stratum, represented by a formula per soil stratum or by a formula for a point in space and/or time. Usually, some combinations apply.

The text suggests that it is important to consider whether a test parameter value needs to be converted to values for the soil parameters required for the calculations.

Representative values of soil parameters should be estimated for each soil stratum. Soil stratification for a calculation method can differ from the actual stratification of the as-found soil.

Multiple sets of representative values for soil properties can be needed for a single calculation, for example because of assumptions made in the calculation method.

A documented calculation method typically includes guidance on selection of the representative value and the corresponding reference scale.

The uncertainties are assigned for both calculation method and the soil parameters.

A priori knowledge should be used. Examples of a priori knowledge include:

a) a thin, continuous failure surface that can be inferred from geological conditions;

b) a known location of a geological fault

c) a lower value of angle of internal friction corresponding to constant volume conditions during shear;

d) a likely low value for residual strength under undrained conditions.

When selecting a representative value, one should consider the tendency for a failure surface to follow the path of least resistance which can cause an apparent reduction in the parameter mean.

Shear strength values are typically chosen so that they do not depend on the dilatancy of the foundation soil, i.e. the tendency of a soil volume to increase (drained case) or the tendency of the pore pressure to decrease or become negative (undrained case) with change in shear stress.

A.5.3.2 Determination of the representative and design values of soil parameters

When determining the representative and the design value (or design resistance), one typically (see also Figure 2):

a) integrates the data from all available sources of information;

b) includes the effect of the uncertainties on the representative value;

c) checks that the representative value applies to the limit state;

d) checks that the representative value is consistent with the calculation method;

e) ensures that the representative value of a soil parameter for a soil stratum corresponds to the volume of soil influencing the occurrence of the limit state under consideration (e.g. potential failure volume or failure surface.

The domain of influence and scale of reference are a function of the characteristics of the soil heterogeneity under consideration.

The selection of a representative value can be based on sensitivity analysis of the effects of uncertain parameters on the results of calculations for verification of a limit state.

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The representative value should be selected according to the conditions that are critical for a limit state. This means that the representative value of, e.g., undrained shear resistance for pile capacity will be lower than that for pile installation, apart from any differences in specific definition of undrained shear resistance required for the calculation models. The same applies to skirt penetration resistance calculations.

A.5.4 Reliability-based geotechnical design

The life of an offshore structure requires decisions of different types and based on different degrees of information, for example for:

a) design with respect to ultimate and serviceability limit states;

b) testing and quality control during planning, design and operation of the offshore structure;

c) planning of inspection and monitoring during installation, operation, repairs or maintenance;

d) decisions on life extension and removal/replacement of an offshore structure.

A reliability-based design (leading to risk-informed decisions) would require that the annual failure probability, given the consequences, does not exceed a target annual failure probability and that the annual reliability index exceeds a target annual reliability index. The target reliability level should be selected as a function of the consequence (loss of life, property and environment) and the nature of a potential loss of structural integrity or functional requirements.

ISO 2394 [175] suggests tentative target reliability levels based on life quality index related to one-year reference period and ultimate limit states. In ISO 2394, life quality index does not include considerations for environmental impact.

For reliability-based design of foundations, a target annual reliability index of 4 (or annual failure probability of $3 \cdot 10^{-5}$) can, for example, be used as a target reliability level [199].

A.5.5 Testing and instrumentation

No additional guidance is offered.

A.6 Guidance on site investigation, identification of geohazards and carbonate soils

A.6.1 General

No additional guidance is offered.

A.6.2 Geological modelling and identification of hazards

No additional guidance is offered.

A.6.3 Carbonate soils

A.6.3.1 General

Carbonate soils cover over 35 % of the ocean floor. For the most part, these soils are biogenic. That is, carbonate soils are composed of large accumulations of the skeletal remains of plant and animal life, such as coralline algae, coccoliths, foraminifera, and echinoderms. To a lesser extent, carbonate soils also exist as non-skeletal material in the form of oolites, pellets, grape-stone, etc. These carbonate deposits are abundant in the warm, shallow water of the tropics, particularly between the 30° north and south latitudes. Deep-sea carbonate oozes have been reported at locations considerably outside these mid-latitudes. Since temperature and water conditions (water depth, salinity, etc.) have varied throughout geological history, ancient deposits of carbonate material can be found buried under more recent terrestrial material outside the present zone of probable active deposition. Major carbonate deposits are known to exist in the Gulf of Mexico along the Florida coastline and in the Bay of Campeche, as well as in the Arabo-Persian Gulf and the Red Sea, in the southern Mediterranean Sea, offshore India and in the north-western Australian shelf.

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The comments in A.6.3.2 to A.6.3.3 focus primarily on carbonate silts and sands. Clay soils with varying proportions of carbonate content are common offshore and a low plasticity index is generally specific to such carbonate clays, but there is little guidance as to how conventional design approaches for clay soils should be modified for different carbonate content. Local experience is important in making such assessments.

A.6.3.2 Characteristic features and properties of carbonate soils

Carbonate soils differ in many ways from silica-rich soils. An important distinction is that the major constituent of carbonate soils is calcium carbonate, which has a low hardness value compared to quartz (the predominant constituent of the silica-rich sediments). Susceptibility of carbonate soils to disintegration (crushing) into smaller fractions at relatively low stress levels is partly attributed to this condition. Typically, carbonate soils have large interparticle and intraparticle porosity, resulting in high void ratio and low density and, hence, are more compressible than soils from a terrigenous silica deposit. Furthermore, carbonate soils are prone to post deposition alterations by biological and physiochemical processes under normal pressure and temperature conditions. This results in the formation of irregular and discontinuous layers and lenses of cemented material. These alterations, in turn, profoundly affect mechanical behaviour.

The fabric of carbonate soils is an important characteristic feature. Generally, particles of skeletal material will be angular to subrounded in shape, with rough surfaces, and have intraparticle voids. Particles of non-skeletal material, on the other hand, are solid with smooth surfaces and without intraparticle voids. It is generally understood that uncemented carbonate soils consisting of rounded non-skeletal grains that are resistant to crushing are stronger foundation materials than carbonate soils that show partial cementation but allow a moderate degree of crushing. There is information that indicates the importance of carbonate content as it relates to the behaviour of carbonate sediments. A soil matrix that is predominantly carbonate is more likely to undergo degradation due to crushing and compressibility of the material than soil that has low carbonate fraction in the matrix. Other important characteristic features that influence the behaviour of the material are grain angularity, initial void ratio, compressibility, and grain crushing. These characteristic features are interrelated parameters in the sense that carbonate soils with highly angular particles often have a high in situ void ratio due to particle orientation. These soils are more susceptible to grain crushing due to angularity of the particles and thus will be more apt to be compressed.

This subclause gives a general overview of the mechanical behaviour of carbonate soils. For a more detailed understanding of material characteristics, information can be found in the proceedings of the specialized international conferences on engineering of calcareous sediments (see References [19] and [7]).

Globally, it is increasingly evident that there is no unique combination of laboratory and in situ testing programme that is likely to provide all the appropriate parameters for design of foundations in carbonate soils. Some laboratory and in situ tests have been found useful. As a minimum, a laboratory testing programme for carbonate soils should determine:

a) material composition, particularly carbonate content;

b) material origin to differentiate between skeletal and non-skeletal sediments;

c) grain characteristics, such as particle angularity, porosity, and initial void ratio;

d) compressibility of the material;

e) soil strength parameters and volume change characteristics on shearing, including effects of cyclic actions;

f) formation cementation, at least in a qualitative sense.

No universally recognized classification system is presently available for carbonate materials. Classification charts for carbonate soils and rocks have been tentatively developed [74] [264] based on grain size, carbonate content and unconfined compressive strength of materials. It is recognized today that parameters such as grain crushability or skeleton compressibility play an important role in assessing the engineering properties of carbonate materials. However, in the absence of a more definite classification scheme, the proposed charts can provide useful guidance.

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A.6.3.3 Foundations in carbonate soils

A.6.3.3.1 Driven piles and other deep foundation alternatives

The current trend for deep foundations in carbonate sands and silts is a move away from driven piles. However, because of lower installation costs, driven piles still receive consideration for support of lightly loaded structures or where extensive local pile loading test data and experience exists to substantiate the design premise. Furthermore, driven piles can be appropriate in moderately competent carbonate soils. At present, the preferred alternative to the driven pile is the drilled and grouted pile. Drilled and grouted piles mobilise significantly higher unit skin friction. The result is a substantial reduction in the required pile penetration compared with driven piles.

Because of the high construction cost of drilled and grouted piles, an alternative driven and grouted pile system has received some attention in the past [34]. This system has the potential to reduce installation costs while achieving comparable capacity, but quality control of the grout injection between the soil and the pile outer wall is the main uncertainty. For any type of grouted pile, the potential for reduction in friction capacity due to cyclic actions should be considered, especially once slip has been initiated between pile and soil.

A.6.3.3.2 Shallow foundations

Carbonate sands and silts generally have higher friction angles than silica sands and silts, but are more compressible, and these two factors influence bearing capacity in opposite ways. Carbonate sands and silts are also generally less permeable than equivalent silica material, leading to longer drainage times for a given size of foundation. The tendency for volume reduction on shearing, particularly under cyclic actions, combined with longer drainage times, leads to potential for bearing failure induced by soil liquefaction. Undrained cyclic strength of carbonate sands is generally lower than for most silica sands. The high compressibility of most carbonate sediments results in relatively large consolidation settlements, and can give rise to large settlements induced by cyclic actions.

A.6.3.3.3 Assessment

Stemming from some publications describing poor foundation performance in carbonate soils and the financial consequences of the remedial measures, there is a growing tendency to take a conservative approach to design in carbonate soils, even if the carbonate content in the sediment fraction is relatively low. This is not always warranted. As with other designs, the judgment of knowledgeable engineering remains a critical link in economic design of offshore foundations in carbonate soil environments.

A.7 Guidance on design of shallow and intermediate foundations for fixed structures

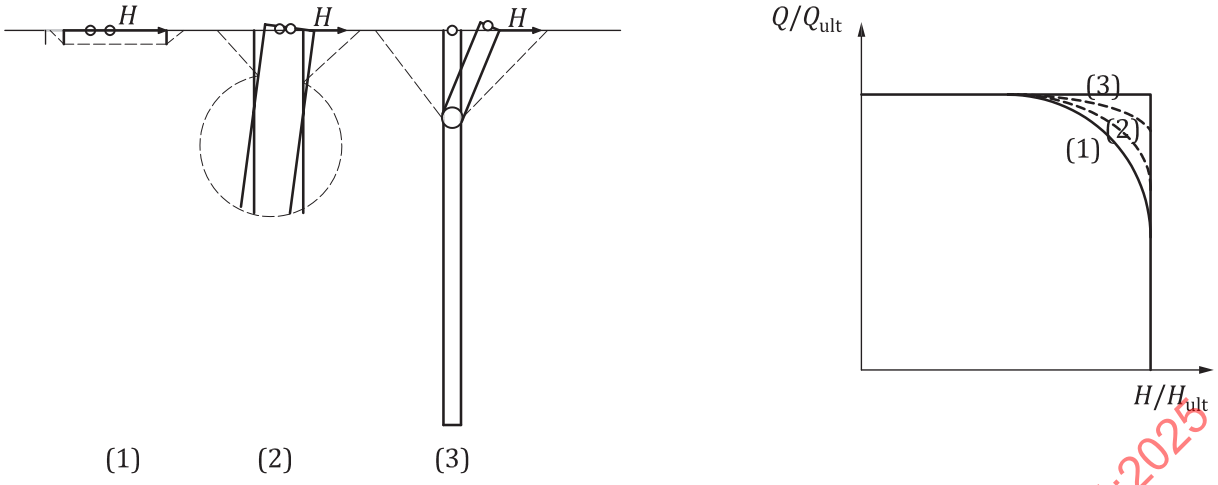
A.7.1 General

In assessing the validity of shallow or intermediate foundation response the actual behaviour of the foundation is most important, which should be assessed by considering aspects such as the foundation dimensions, soil conditions, installation method and applied loading. A shallow foundation is typically designed considering 'surface' failure mechanisms, i.e. failure mechanisms that reach the seafloor while an intermediate foundation can be designed against mechanisms that involve both surface and confined (below the seafloor) aspects. Idealized failure mechanisms and action interaction for shallow foundations, intermediate foundations and pile foundations are illustrated in Figure A.1. The simplified shallow and intermediate design methodology presented here typically assumes rigid foundation behaviour.

Guidance in this clause relates to foundations for fixed structures, whether subsea or surface piercing. Examples include shallow and intermediate foundations for surface piercing concrete gravity-based structures, steel jacket structures, minimum structures, monotowers; and shallow and intermediate foundations for subsea structures, including manifolds, overtrawl structures, inline structures, pipeline end termination structures and holdback anchors for subsea structures. Foundations are considered in Clause 10 for riser towers and in Clause 11 for floating. A range of structures for which the shallow and intermediate foundations considered in this clause can apply are illustrated in Figure A.2. Foundations in this clause can be temporary (e.g. a construction aid during piling) or the main permanent foundation.

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a) Failure mechanisms **b) Loading interaction**

Key

1 shallow foundation
2 intermediate foundation
3 pile foundation, excluding H-Q interaction

Figure A.1 — Illustration of of shallow foundations, intermediate foundations and flexible piles

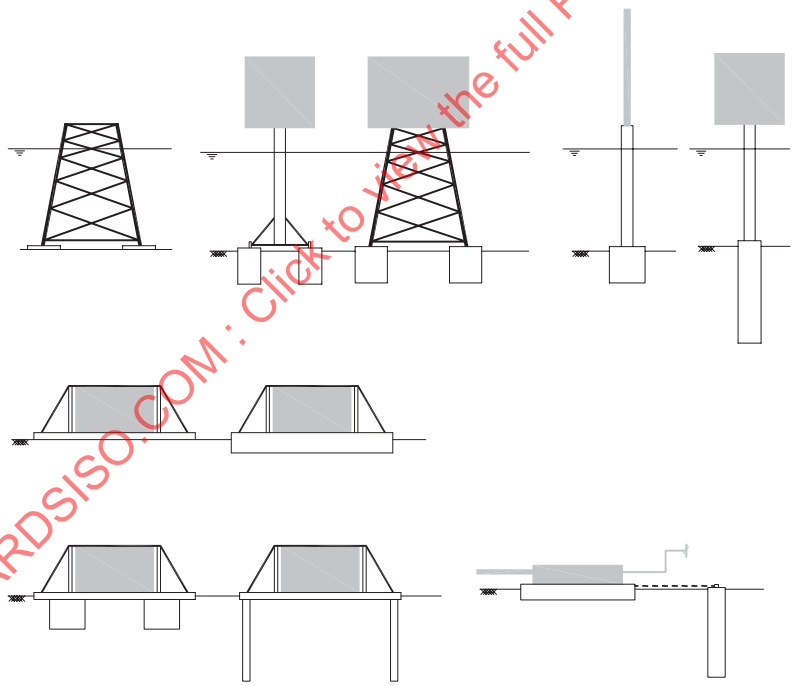


Figure A.2 — Illustration of foundation types and applications

The calculation approaches presented in this document for evaluating the installation resistance, ultimate limit state (stability) and serviceability limit state (displacement) of shallow and intermediate foundations given are based on solutions for simple soil profiles and idealized soil response, i.e. uniform or linearly increasing strength or stiffness with depth and fully drained or undrained soil response. The formulae should only be applied to conditions similar to those for which they were derived or for which they can be shown to be applicable.

The calculation approaches presented are limited in nature and are not necessarily appropriate for design in a number of situations.

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In circumstances such as these, general guidance cannot be provided and reliance should be placed on experience, published case histories, physical testing and numerical modelling.

A.7.2 Principles

A.7.2.1 General principles

No additional guidance is offered.

A.7.2.2 Foundation embedment

No additional guidance is offered.

A.7.2.3 Sign conventions, nomenclature and action reference point

For an embedded foundation, the actions on the top of the foundation are transferred to the foundation base level (tip of skirt for a skirted foundation). This is done by modifying the factored actions applied to the top of the foundation to account for:

a) soil resistance on the sides of the embedded foundation;

b) submerged foundation weight;

c) submerged soil weight within skirts, if not counteracted by the exterior soil or otherwise applicable.

Action factors should be applied to the foundation weight and soil weight, considering the impact of the action factor on resultant resistance.

The soil resistance on the sides of the embedded foundation consists of:

— horizontal passive and active soil resistance;

— frictional resistance on the skirts.

Frictional resistance on the skirts can reduce the vertical and moment action transmitted to the underlying soil, and can be considered in the design. Specific guidance is not provided here.

A.7.3 Acceptance criteria

A.7.3.1 Material and action factors

Refer to A.7.7.2 for further performance-based design guidance.

A.7.3.2 Use of partial factors in design

Applying the material factor as a resistance factor on the soil resistance envelope derived with unfactored representative soil properties, can be attractive in interbedded sand and clay profiles ore when cyclic loading is an important design consideration.

A.7.4 Design considerations

A.7.4.1 Adjusting for soil plug weight

No additional guidance is offered.

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A.7.4.2 Skirt spacing

Skirts can be used on subsea foundations to:

— develop sufficient sliding capacity (especially as seafloor strengths are often low, and difficult to measure);

— protect foundation performance against undermining due to scour.

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

— use of internal and external skirts to the extent that the sliding failure mechanism is that of skirt tip sliding;

— acceptance of failure mechanism rising above skirt tip level, if the combination of sliding resistance inside the compartment and flow around the embedded members is sufficient to satisfy overall sliding capacity requirements.

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

References [1] and [124] provide guidance on skirt spacing for simplified seabed conditions.

A.7.4.3 Foundation base perforations

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

a) assist in loading out the structure through the splash zone without dynamically overloading of the associated load-out rigging;

b) assist the approach of the structure to the seabed without hydraulic/hydrodynamic instability;

c) assist in preventing the seabed from being unduly impacted by scour or local bearing failure.

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

Specific guidance is not provided on the ratio or distribution of top plate perforations.

Reference [322] provides guidance on the impact of perforations on foundation vertical capacity.

A.7.4.4 Skirtless foundations penetrating soft soils

Allowable differential settlement of the structure will depend upon the type of structure and its installation, and should be subject of a risk assessment. Adequate precautions should be taken to minimize differential settlements between foundations. Good practice should ensure that there is an acceptable limit to the penetration.

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

A.7.4.5 Tensile stresses beneath foundations

No additional guidance is offered.

A.7.4.6 Omni-directional loading

Refer to 7.7.1 and A.7.7.1.

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A.7.4.7 Interaction with other structures

Interaction with spudcans is addressed in ISO 19905. Interaction with conductors is addressed in 10.5.2.

A.7.4.8 Multiple foundations

In many instances, use of multiple shallow foundations can significantly increase overall foundation capacity, as illustrated in References [148] and [188].

A.7.4.9 Hydraulic stability

A.7.4.9.1 Scour

No additional guidance is offered.

A.7.4.9.2 Piping

No additional guidance is offered.

A.7.4.10 Unconventional soils or soil profiles

The methods outlined are strictly applicable to conditions of uniformly varying soil strength, although reasonable assessment of equivalent uniform properties can frequently be made. Attention should be given to defining the soil parameters throughout the expected zone of influence, encompassing the zone of influence from all actions.

The use of the standard stability formulae presented in this annex are not applicable where:

a) soil conditions are heterogeneous or anisotropic;

b) loading conditions deviate from the conditions assumed in the bearing capacity formulae;

c) loading rates are such that the conditions are partially drained; or

d) foundation geometries are highly irregular.

In those cases, alternative procedures such as one or combinations of the following should be selected:

— use of limit analysis to determine bounds on failure actions and to determine relative sensitivity of failure actions to parameters of interest;

— use of numerical analysis to solve the governing formulae directly;

— use of scaled model tests to verify calculation models and procedures.

References [73] and [370] provide advice on the use and design of subsea gravel and rock fills.

References [35] and [354] provide advice on rock-steel interface properties.

Refer to 7.5.1.6 and A.7.5.1.6 for surficial crusts.

Refer to A.6.3.3.2 for shallow foundations on carbonate soils.

A.7.4.11 Selection of soil parameters for design

A.7.4.11.1 Parameters used in serviceability design

The following general practices are recommended.

a) For strongly dilatant soils, high undrained shear strengths can be used in design only if the potential loss of dilatancy on shear surfaces has been explicitly addressed.

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b) In soft and very soft clays, lab vane (on extruded samples), pocket penetrometer, torvane, fall cone, unconsolidated undrained triaxial tests and unconfined compression tests are unreliable and should not be used. Consolidated undrained triaxial tests with pore pressure measurement, simple shear tests, in situ vane, and penetrometer, ball or T-bar tests (where the correlation between penetration resistance and soil strength is known for a particular soil) are more reliable techniques and should be used for determining undrained shear strength of soft and very soft clays.

c) Soils display undrained shear strength anisotropy, and thus triaxial compression, triaxial extension and simple shear strengths can be different. Care should be taken to adopt an appropriate strength in assessing foundation capacity, and any assumptions made in this regard should be clearly documented.

d) For drained bearing capacity calculations for sands, the effective plane strain angle of friction should be used, which is generally 10 % higher than that measured in a triaxial compression test. This value should be determined at the appropriate stress level.

e) Foundation stability under cyclic loading conditions can be assessed using pseudo-static analysis provided appropriately derived cyclic shear strengths are used. One approach for deriving appropriate cyclic soil strengths for use in pseudo-static stability analysis, takes account of action history and average shear stress. A procedure for how this can be done is presented in Reference [1].

f) In many instances, cyclic performance of non-cohesive soils can be assessed using cyclic undrained soil strengths derived in a manner similar to that used for cohesive soils. A procedure for how this can be done is presented in Reference [10]. In undertaking such analyses, it is important to account for the effects of drainage with the potential for dissipation of excess pore pressures to occur over the duration of cyclic loading.

g) The strain rate at which testing is performed can impact the observed result, and rate effects should be considered when assessing foundation response to rapid loading events.

h) Where applicable, the effect of soil consolidation on strength can be considered in design. This typically increases overall foundation capacity. However, in the case of preloaded foundations, the soil strength enhancement is generally limited to soils directly below the foundation base.

i) In soils that display strain softening behaviour, strain compatibility can impact the selection of soil strength. For example, for skirted foundations, contributions to capacity come from a combination of base shear (at skirt tip level) and passive resistance, which can be mobilised at very different strain levels.

A.7.4.11.2 Wave-seabed interaction

No additional guidance is offered.

A.7.5 Ultimate limit state (stability)

The bearing capacity factors used herein are considered those most commonly used, but alternative factors are available and can be applied at the discretion of the designer. Selection of alternative factors should be justified in design documentation.

A.7.5.1 Assessment of bearing capacity of shallow foundations

The development of the bearing capacity formulae presented assumes that the soil is a rigid, perfectly plastic material that obeys the Mohr-Coulomb yield criterion with associated flow.

The following bearing capacity factors and correction factors primarily come from References [55] and [325].

Analytically precise solutions of bearing capacity factors for perfectly plastic materials (with associated flow) can now be determined by select software programs. Notably, the freeware ABC [228] is based on the method of stress characteristics that calculates lower bound solutions for the vertical bearing capacity of surface strip and circular foundations, with a smooth or rough foundation-soil interface, with or without a surface surcharge.

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A.7.5.1.1 Failure mechanisms

No additional guidance is offered.

A.7.5.1.2 Action transfer

No additional guidance is offered.

A.7.5.1.3 Idealization of foundation area and the effective area concept

Limiting equilibrium methods are generally based on a two-dimensional model (vertical slice) and three-dimensional effects are included by defining the resistance of the vertical side areas. For irregular foundation shapes, this requires a rectangular idealization of the foundation area. This idealized area can be defined by a rectangle of width B and length L having the same area, A, and the same areal moments of inertia, I_x and I_y, as for the real area:

$$A_{\text{idealized}} = BL = A_{\text{real}} \tag{A.1}$$

$$I_{x,\text{idealized}} = I_{x,\text{real}} \text{ for action effects in the y-direction} \tag{A.2}$$

$$I_{y,\text{idealized}} = I_{y,\text{real}} \text{ for action effects in the x-direction} \tag{A.3}$$

The width, B, and length, L, of the idealized foundation area are determined by solving Formulae (A.1), (A.2) and (A.3).

Action eccentricity decreases the ultimate vertical action that a shallow foundation can withstand. This is accounted for in bearing capacity analysis by reducing the effective area of the foundation.

Figure A.3 illustrates shallow foundations with eccentric actions. The eccentricity, e, is the distance from the centre of a shallow foundation to the point of action of the resultant, measured parallel to the plane of the soil-foundation contact. The point of action of the resultant is the centroid of the reduced area. The distance e is M/Q, where M is the applied overturning moment and Q is the vertical action. The action factors defined in ISO 19902 for beneficial effects of actions should be used to assess the design vertical action when deriving the eccentricity due to moment loading.

Where a skirted foundation incorporates a sealed base plate and for undrained analyses, the vertical action used to calculate the effective area may include a contribution due to soil trapped within the skirted area. The following is noted:

a) Drained foundation analysis based on the effective area method should exclude the weight of the soil plug (i.e. the soil trapped within the skirts).

b) Submerged soil weight should be used for calculating soil plug contribution. It should be ensured that inclusion of the soil plug does not lead to less conservative foundation design.

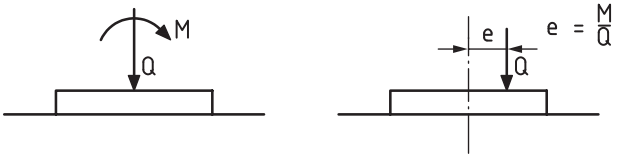
For a rectangular base area Figure A.3 b), eccentricity can occur with respect to either axis of the foundation. A simplified means of addressing this is to reduce the dimensions of the foundation in both directions:

$$\begin{aligned} L' &= L - 2e_1 \\ B' &= B - 2e_2 \end{aligned} \tag{A.4}$$

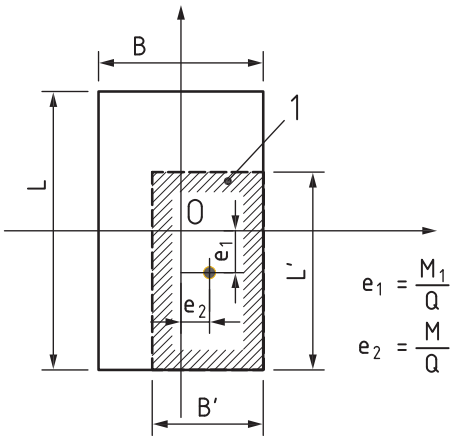
With $B' \leq L'$ and where L and B are the foundation length and width, respectively, the prime denotes effective dimensions, and e_1 and e_2 are eccentricities along the length and width.

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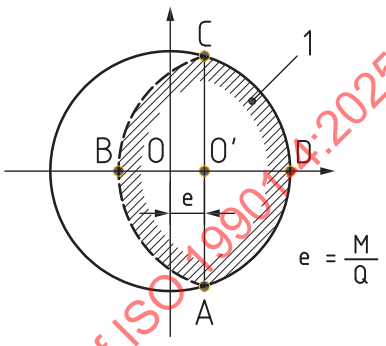
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a) Equivalent loadings



b) Reduced area — Rectangular foundation



c) Reduced area — Circular foundation

Key

1 reduced area

e M/Q eccentricity due to moment M, about axis parallel to A-C

e_1 M_1/Q eccentricity due to moment $M1$, about axis parallel to width, B

e_2 M_2/Q eccentricity due to moment M_2, about axis parallel to length, L

Figure A.3 — Definition of effective area for various foundation geometries

Circular foundations subject to eccentric actions can be idealized as rectangular foundations by solving Formulae (A.1), (A.2) and (A.3). Alternatively, for a circular base with radius, R, the effective area can be assumed as shown in Figure A.3 c). The centroid of the effective area is displaced a distance, e, from the centre of the base. The effective area is then considered to be twice the area of the circular segment ADC.

In addition, the effective area is considered to be rectangular with a length to width ratio equal to the ratio of line lengths AC to BD. The effective dimensions are therefore:

$$A' = 2s = B'L'$$

$$L' = \left(2s\sqrt{\frac{R+e}{R-e}}\right)^{1/2} \tag{A.5}$$

$$B' = L'\sqrt{\frac{R-e}{R+e}}$$

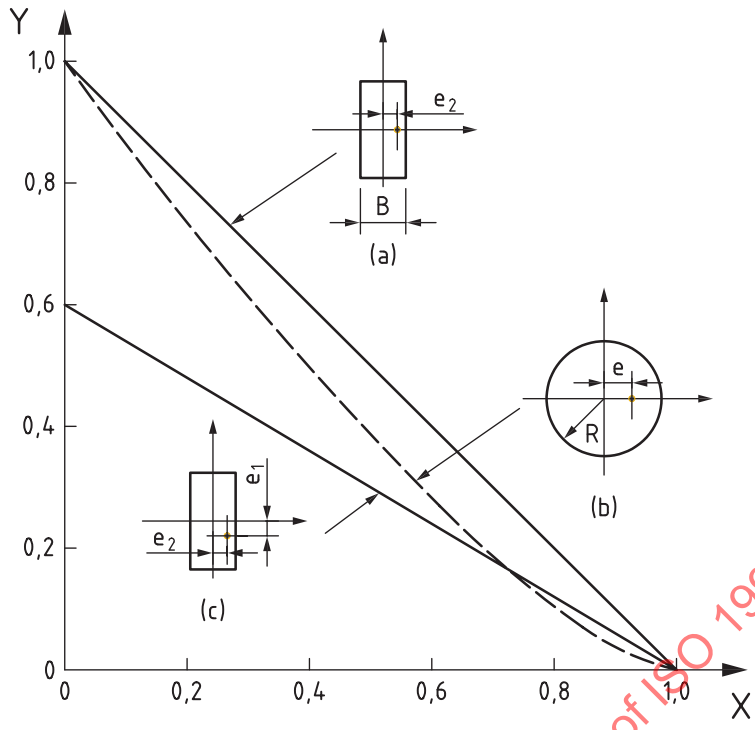
where

$$s = \frac{\pi R^2}{2} - \left(e \times \left(\sqrt{R^2 - e^2}\right) + R^2 \arcsin\left(\frac{e}{R}\right)\right)$$

Examples of effective areas as a function of eccentricity are shown in Figure A.4 in a dimensionless form.

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Key

X dimensionless eccentricity $2e_2/B$, e/R

Y dimensionless reduced area A'/A

a rectangular – 1 Way loading, $e_1/L = 0$

b circular

c rectangular – 2 Way loading, $e_1/L = 0{,}2$

Figure A.4 — Area reduction factors for eccentrically loaded shallow foundations

No published data are available on other foundation shapes. Approximations can be made to find an equivalent rectangular or circular foundation when non-standard shapes are encountered. For example, guidance for triangular shaped foundations is given in Reference [158]. Alternatively, an idealized rectangular foundation can be determined by solving Formulae (A.1), (A.2) and (A.3).

Alternative methods exist for assessing the effect of eccentricity in multiple or non-orthogonal directions [159] and these can be more applicable in more complex conditions.

A.7.5.1.4 Undrained conditions with constant shear strength with depth

A.7.5.1.4.1 Bearing capacity factors

The bearing capacity factor, N_c, for a rigid surface strip foundation with horizontal base resting on the surface of a horizontal seafloor, idealized as a perfectly plastic material of uniform strength under uniaxial vertical action in the absence of other actions is given by Reference [269].

$$N_c = 2 + \pi = 5{,}14 \quad \text{(A.6)}$$

Correction factors are applied to extend the basic bearing capacity solution to account for inclined actions, foundation shape, depth of embedment, foundation base inclination and seafloor surface inclination.

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A.7.5.1.4.2 Bearing capacity correction factors

For cases of constant isotropic undrained shear strength with depth, the following bearing capacity correction factors are recommended:

$$K_c = 1 + s_c + d_c - i_c - b_c - g_c \tag{A.7}$$

where s_c, d_c, i_c, b_c, and g_c are correction factors related to foundation shape, embedment depth, action inclination, base inclination and seafloor surface inclination, respectively, where:

$$s_c = 0{,}18(1-2i_c)\left(\frac{B'}{L'}\right) \tag{A.8}$$

$$d_C = 0{,}3\arctan\left(\frac{D_b}{B'}\right) \tag{A.9}$$

$$i_c = 0{,}5 - 0{,}5\sqrt{1-\frac{H_b}{A'\left(\frac{s_{u0}}{\gamma_m}\right)}} \tag{A.10}$$

$$b_c = \frac{2\upsilon}{\pi+2} \approx 0{,}4\upsilon \tag{A.11}$$

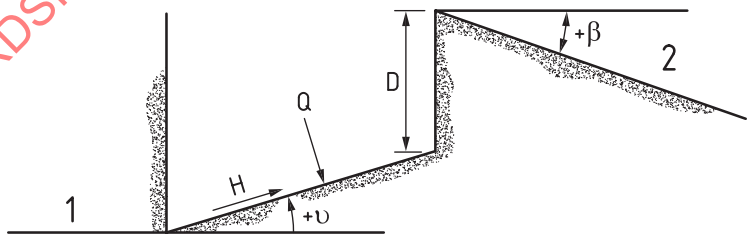
$$g_c = \frac{2\beta}{\pi+2} \approx 0{,}4\beta \tag{A.12}$$

where

a) The effective width or effective length is used for action eccentricity parallel to the width or length. The effective width and effective length are used for orthogonal eccentric actions parallel to the width and length. B' and L' are determined from Formulae (A.1), (A.2) and (A.3).

b) H_b refers to the factored action applied to the effective area component of the base only. This corresponds to the total lateral action applied to the foundation minus any soil resistance acting on the foundation above skirt tip level, and minus any lateral resistance that can be carried by shearing at skirt tip level outside the effective area. The material factor for pure sliding conditions should be applied to these two resistance components prior to them being subtracted from the total lateral action.

c) υ and β are base and ground inclination angles in radians. Figure A.5 defines these angles for a general foundation problem.



Key

1 horizontal surface

2 ground surface

Figure A.5 — Definitions for inclined base and seafloor surface [55]

The recommended correction factors s_c, i_c, b_c, and g_c are taken directly from Reference [55].

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The recommended depth factor d_c is slightly more conservative than specified by Reference [55]. The relevancy of using the above depth factor d_c should be evaluated for individual cases. If the installation procedure or other foundation aspects, such as scour, do not allow for the required mobilization of shear stresses in the soil above foundation base level, it is recommended that $d_c = 0$. In addition, it is recommended that $d_c = 0$, if the horizontal action leads to high utilisation of passive earth pressure between seafloor and foundation base level.

A.7.5.1.5 Undrained conditions with linearly increasing shear strength with depth

For cases of linearly increasing isotropic undrained shear strength with depth, the following correction factors F and K_c are recommended.

F is an empirical value taken as a function of $\kappa B'/s_{u0}$ and further discussed in Reference [95].

In selection of F, rough conditions can generally be adopted for unpainted skirted foundations. Values of F can be approximated using the relationship:

$$F \approx a + bx - \sqrt{(c+bx)^2 + d^2} \tag{A.13}$$

where

x = $\kappa B'/s_{u0}$ and is valid for $0 \le x \le 25$;

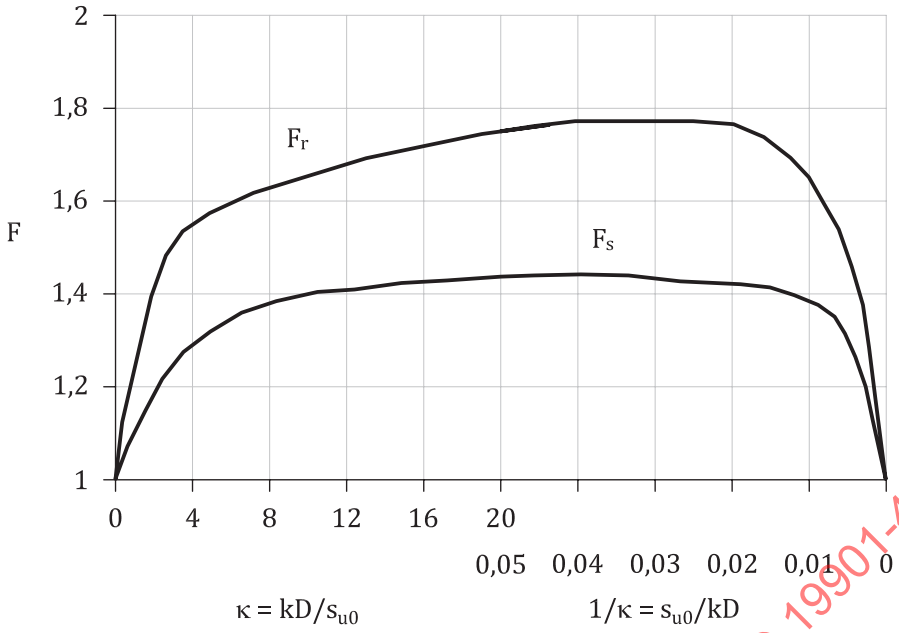
a, b, c, and d are constants that vary with roughness and are outlined in Table A.1.

Table A.1 — Modification factors for soil strength heterogeneity (see Figure A.6)

Constant	Fully rough interface	Fully smooth interface
-	F_r	F_s
a	2,560	1,372
b	0,457	0,070
c	0,713	−0,128
d	1,380	0,342

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Key

F_s is for no friction at soil–foundation interface ("smooth" foundation)

F_r is for friction equal to the shear strength of soil at the interface ("rough" foundation)

F bearing capacity correction factor

Figure A.6 — Bearing capacity correction factor *F* for linearly increasing isotropic undrained shear strength with depth [95]

$$K_c = 1 + s_c + d_c - i_c - g_c - b_c \tag{A.14}$$

where s_c, d_c, i_c, b_c, and g_c are correction factors related to foundation shape, embedment depth, action inclination, base inclination and seafloor surface inclination respectively, as further detailed hereafter:

$$s_c = s_{cv}\,(1 - 2i_c)\left(\frac{B'}{L'}\right) \tag{A.15}$$

where s_{cv} is taken as function of $\kappa B'/s_{u0}$ (see Table A.2) and values of s_{cv} can be approximated using the relationship:

$$s_{cv} \approx 0{,}18 - 0{,}155\sqrt{x} + 0{,}021x \tag{A.16}$$

where $x = \kappa B'/s_{u0}$ and is valid for $0 \le x \le 10$.

Table A.2 — Shape factor coefficients for circular or square foundations under pure vertical actions

$\kappa B'/s_{u0}$	s_{cv}
0	0,18
2	0,00
4	-0,05
6	-0,07
8	-0,09
10	-0,10

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$$d_c = 0{,}3\left(\frac{s_{u,1}}{s_{u,2}}\right)\arctan\left(\frac{D_b}{B'}\right) \tag{A.17}$$

where

$s_{u,1}$ is the average shear strength above base level;

$s_{u,2}$ is the equivalent shear strength below base level, given by:

$$s_{u,2} = F\frac{\left(N_c s_{u0} + \frac{\kappa B'}{4}\right)}{N_c} \tag{A.18}$$

$$i_c = 0{,}5 - 0{,}5\sqrt{1 - \frac{H_b}{A'\left(\frac{s_{u0}}{\gamma_m}\right)}} \tag{A.19}$$

$$b_c = \frac{2\upsilon}{\pi + 2} \approx 0{,}4\upsilon \tag{A.20}$$

$$g_c = \frac{2\beta}{\pi + 2} \approx 0{,}4\beta \tag{A.21}$$

The effective width or effective length is used for action eccentricity parallel to the width or length. The effective width and effective length are used for orthogonal eccentric actions parallel to the width and length. B' and L' are determined from Formulae (A.1) to (A.3).

H_b, υ and β are as noted under A.7.5.1.4.2.

The recommended d_c is based upon the correction factor for constant isotropic undrained shear strength, but modified to account for linearly increasing undrained shear strength with depth.

The shape factor, s_{cv}, from Reference [299] for axial symmetry and pure vertical action, is assumed to be approximately valid for an equivalent square foundation (B'/L') = 1.

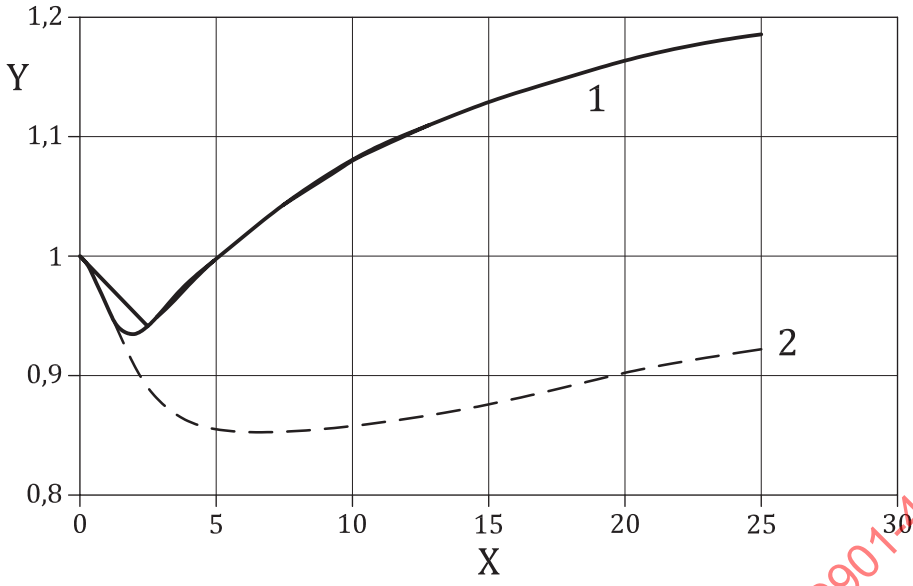
The relevancy of using the depth factor, d_c, should be evaluated in each case. If the installation procedure or other foundation aspects, such as scour, do not allow for the required mobilisation of shear stresses in the soil above foundation base level, it is typical to assume d_c = 0. In addition, the mobilization of passive earth pressure between seafloor and foundation base level can reduce dc

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

The crust correction factor, F_c, may be evaluated from Figure A.7 after Reference [95].

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Key

X normalized undrained strength kB'/s_{uC}
Y bearing capacity correction factor, F_c
1 fully rough interfaces between foundation and seabed
2 fully smooth interfaces between foundation and seabed

Figure A.7 — Bearing capacity correction factor F_c for surficial crust

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A.7.5.1.7 Drained conditions

A.7.5.1.7.1 Bearing capacity factors

The bearing capacity factors given in Formulae (A.22) and (A.23) are recommended for pure vertical action on a strip foundation with no embedment, for which Reference [226] provides exact formulations:

$$N_q = \left(\tan\left(\frac{\pi}{4} + 0{,}5 \arctan\left(\frac{\tan\phi'}{\gamma_m} \right) \right) \right)^2 \left(\exp\left(\pi \frac{\tan\phi'}{\gamma_m} \right) \right) \tag{A.22}$$

$$N_\gamma = 1{,}5\left(N_q - 1\right)\left(\frac{\tan\phi'}{\gamma_m} \right) \tag{A.23}$$

Effective plane-strain friction angles, ϕ', between 30° and 42° are considered typical limits for general use with Formulae (A.22) and (A.23). Friction angles that fall outside of these limits can indicate non-standard soils or poor quality laboratory testing, especially when falling below 30°.

A.7.5.1.7.2 Bearing capacity correction factors

For drained conditions, the following bearing capacity correction factors are recommended:

$$K_q = s_q d_q i_q b_q g_q \tag{A.24}$$

$$K_\gamma = s_\gamma d_\gamma i_\gamma b_\gamma g_\gamma \tag{A.25}$$

where s, d, i, b and g are correction factors related to foundation shape, embedment depth, action inclination, base inclination and seafloor surface inclination, respectively. The subscripts q and γ identify the bearing capacity factor, N_q or N_γ, with which the correction term is associated.

The factors given in Formulae (A.26) to (A.34) for seafloor surface inclination can be unconservative in cases of loose to very loose sand.

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Recommended expressions for the correction factors are:

$$s_q = 1 + i_q \left(\frac{B'}{L'} \right) \sin\left(\arctan\left(\frac{\tan\phi'}{\gamma_m} \right) \right) \quad \text{(A.26)}$$

$$d_q = 1 + 1{,}2 \left(\frac{D_b}{B'} \right) \left(\frac{\tan\phi'}{\gamma_m} \right) \left(1 - \sin\left(\arctan\left(\frac{\tan\phi'}{\gamma_m} \right) \right) \right)^2 \quad \text{(A.27)}$$

$$b_q = \mathrm{e}^{-2v\left(\frac{\tan\phi'}{\gamma_m} \right)} \quad \text{(A.28)}$$

$$g_q = g_\gamma = (1 - 0{,}5\tan\beta)^5 \quad \text{(A.29)}$$

$$i_q = \left[1 - 0{,}5 \left(\frac{H_b}{V_b} \right) \right]^5 \quad \text{(A.30)}$$

$$s_\gamma = 1 - 0{,}4 i_\gamma \left(\frac{B'}{L'} \right) \quad \text{(A.31)}$$

$$d_\gamma = 1 \quad \text{(A.32)}$$

$$i_\gamma = \left[1 - 0{,}7 \left(\frac{H_b}{V_b} \right) \right]^5 \quad \text{(A.33)}$$

$$b_\gamma = \mathrm{e}^{-2{,}7v\left(\tan\frac{\phi'}{\gamma_m} \right)} \quad \text{(A.34)}$$

The effective width or effective length is used in the bearing capacity correction factors stated in Formulae (A.26) to (A.34) for action eccentricity parallel to the width or length. The effective width and effective length are used for orthogonal eccentric actions parallel to the width and length.

The relevancy of using the depth factor, d_q, should be evaluated in each case. The effect of foundation embedment is very sensitive to soil disturbance at the soil-structure interface along the sides of the embedded base. If the installation procedure or other foundation aspects, such as scour, do not allow for the required mobilization of shear stresses in the soil above foundation base level, it is recommended that $d_q = 1{,}0$. It is further recommended that $d_q = 1{,}0$ if the horizontal action leads to mobilization of significant passive earth pressure between the seafloor and foundation base level.

H_b, v and β are as noted in A.7.5.1.4.2.

A.7.5.1.7.3 Exclusion of effective cohesion from bearing capacity formulae

The effective strength envelope for a given soil is often quoted in terms of a 'cohesion intercept', c', and effective friction angle, ϕ', with the envelope fitted to results of laboratory tests conducted at different levels of effective confining stress.

It is debatable whether the deduced c' reflects a true cohesion (or cementation) or is an artefact resulting from fitting a tangent to what is, in reality, a curved strength envelope because the friction angle for soils increases as the mean effective stress level decreases, due to increasing dilation. As such, the effective cementation maybe as an artificial quantity arising from the interpretation of the laboratory tests rather than a true physical quantity.

However, there is considerable debate over this among geotechnical specialists as documented in Reference [301].

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Examples of where an effective cohesion can be included in estimating bearing capacity include:

a) Naturally cemented soils (particularly sands)

noting however that the cementation can reduce to zero before the full frictional strength is mobilised.

b) Medium to heavily over-consolidated clays

In this case, ignoring any effective cohesion (or dilation-induced high friction angles at low mean effective stresses) can possibly prove over-conservative.

A.7.5.2 Assessment of sliding capacity of shallow foundations

A.7.5.2.1 General

No additional guidance is offered.

A.7.5.2.2 Undrained conditions

No additional guidance is offered.

A.7.5.2.3 Drained conditions

No additional guidance is offered.

A.7.5.2.4 Horizontal seabed resistance above foundation base level

Contributions to horizontal resistance of an embedded foundation can come from:

— base shear;

— soil resistance above skirt tip level due to the difference between active and passive resistance;

— side shear on members located parallel to the direction of lateral loading.

The amount of side shear that can be adopted in the design is a function of the shearing on the interface between the embedded member and the soil, and can be influenced by soil disturbance during installation and scour. Specific guidance is not provided in relation to calculating side shear.

In regard to active and passive resistance, the following guidance is provided:

a) Undrained conditions

The undrained horizontal soil reaction coefficient K_{ru} depends on several factors, such as roughness, foundation shape, side shear, depth of embedment, and possible side gap between foundation and soil due to installation or from scour.

A value of K_{ru} = 4 is recommended for cases in which both active and passive resistance can be relied upon and significant scour is not expected.

A value of K_{ru} = 2 is recommended for cases in which active soil pressures do not develop, such as due to cracking or installation disturbance, and significant scour is not expected on the passive side of the foundation. In this case, the weight of soil within the passive soil wedge can be accounted for. In such cases, the total lateral resistance calculated should not exceed that which will be calculated using K_{ru} = 4.

b) Drained conditions

The drained horizontal soil reaction factor K_{rd} depends on several factors, such as mobilised soil friction angle, roughness, foundation shape, side shear, depth of embedment, and possible side gap between foundation and soil from installation or from scour. Formula (A.35) is recommended.

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$$K_{rd} = K_p - \left(\frac{1}{K_p} \right) \tag{A.35}$$

where K_p is the passive earth pressure coefficient and is given by:

$$K_p = \left(\tan\left(\frac{\pi}{4} + 0{,}5 \arctan\left(\frac{\tan\phi'}{\gamma_m} \right) \right) \right)^2 \tag{A.36}$$

A.7.5.2.5 Assessment of torsional capacity

Methods that can be used for assessing torsional stability are provided in References [127], [126] and [65].

A.7.5.3 Assessment of capacity of intermediate foundations

No additional guidance is offered.

A.7.6 Serviceability limit state (displacements and rotations)

A.7.6.1 General

No additional guidance is offered.

A.7.6.2 Serviceability of shallow foundations under static loading

A.7.6.2.1 General

Static deformations (displacements and rotations) are generally considered to be of the following two types:

a) immediate deformation, primarily resulting from shear deformation (shear straining) of the soil;

b) long-term deformation, which occurs over a period of time and is primarily associated with a gradual dissipation of excess pore pressures and associated volume changes of the soil (i.e. primary consolidation).

In addition, secondary displacement due to creep can occur.

A.7.6.2.2 Immediate displacements and rotations

Because soils exhibit nonlinear, loading history dependent behaviour, the short-term deformation problem is complex. For monotonic, low level actions (with respect to failure actions) estimates of deformation can be made assuming the soil to be a homogeneous linearly elastic material.

Solutions for conditions other than those given in 7.5.2.2, including point displacements within the soil mass itself, can be found in Reference [267]. Solutions for rigid, embedded circular foundations are provided in Reference [108].

Considerable care should be exercised in determining the elastic constants of the soil since the elastic moduli of soils depend strongly on the magnitude of effective mean stress and the level of strain. This is particularly significant for highly permeable granular soils where equivalent moduli should be selected from some weighted average mean stress taken over the volume of soil subjected to significant stresses. For relatively impermeable soils, such as clays, a correlation of modulus with strength and over-consolidation ratio can lead to satisfactory results. Further discussion of these points is presented in Reference [165].

Where the foundation base is flexible or the loading is sufficiently severe to create high stresses throughout a significant volume of soil, the formula provided in this document are inappropriate and numerical analyses can be required. Finite element and finite difference techniques have the capability of including complex geometries and loadings and nonlinear, variable soil profiles. The potential effects of softening of the soil (reduction in modulus) under cyclic loading should be considered.

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A.7.6.2.3 Primary consolidation settlement

Because of the finite extent of the foundation, the vertical stress imposed by the structure is attenuated with depth. This attenuation can be estimated from elastic solutions, such as those cited by Reference [267]. This approximate method is particularly appropriate where settlement is governed by thin, near-surface layers.

Rate of settlement is governed by rate of drainage and compressibility. Many soil mechanics textbooks set out methods for one-dimensional consolidation solutions, but in many cases the one-dimensional approximation for flow and strain is unrealistic. Elastic solutions for three-dimensional consolidation settlement around embedded circular foundations are provided in References [145] and [146]. If an accurate prediction of rate of settlement is required, 2D or 3D coupled analysis supported by high quality field data are required.

A.7.6.2.4 Secondary compression: creep

No additional guidance is offered.

A.7.6.2.5 Differential displacements and rotations

No additional guidance is offered.

A.7.6.3 Serviceability of intermediate foundations

No additional guidance is offered.

A.7.6.4 Serviceability in response to dynamic and cyclic actions

In many cases, loading can be considered pseudo-static and the foundation can be treated as an elastic half space subject to the restrictions outlined in 7.5. Consequently, the stiffness of the soil can be calculated in a manner similar to that presented for static conditions.

In some cases, it is not appropriate to treat the foundation as an elastic half space, such as where:

a) soil conditions are not uniform;

b) soil non-linearity is expected at the serviceability loading level;

c) it is necessary to model the energy loss in the soil.

A special case of environmental loading is the response of offshore foundations to cyclic loading arising from waves. The various displacement components and how they can be evaluated are discussed in Reference [1].

A.7.7 Alternative methods of design

A.7.7.1 Yield surface approach

A.7.7.1.1 General

While it is generally accepted that the conventional approach to shallow foundation design using the effective area method is conservative where large horizontal action and overturning moment act, using the effective area method can lead to under-prediction of capacity for some loading situations (e.g. References [143], [147] and [332]).

Fully encompassing yield surface in Q, H, M and T space can be used to predict design loading combinations to reach ultimate limit state, as well as an explicit indication of the effect of a change in individual design action components on proximity to an ultimate limit state. The yield surface method can also be extended to define the action–displacement response of a foundation if used in conjunction with a flow rule. In undrained conditions, when normality can be assumed, the flow rule can be directly derived from the yield surface.

Th yield surface approach is described further in A.7.7.1.2 to A.7.7.1.5.

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A.7.7.1.2 Application to design

The general procedure for developing a yield surface for use in design involves the following.

a) Defining the 'uniaxial' ultimate limit states Q_{ult} ($H = M = 0$), H_{ult} ($M = 0$; and $Q = 0$ where tensile stresses are allowed beneath the foundation, or $Q = Q_{ult}/2$ if no tensile stresses permitted) and M_{ult} ($H = 0$; and $Q = 0$ where tensile stresses are allowed beneath the foundation, or $Q = Q_{ult}/2$ if no tensile stresses permitted) to define the apex points of the yield surface.

b) Defining the shape of the interaction diagram through an expression as a function of (Q/Q_{ult}, H/H_{ult}, M/M_{ult}).

The magnitude of the uniaxial capacity and the shape of the yield surface depend on the soil response to loading (undrained or drained), the soil strength profile (uniform or heterogeneous), foundation shape, foundation embedment, structural connection between adjacent foundations, and tension capacity (or not) between the foundation and the soil.

When adopting a yield surface approach, material factor, γ_m, should be applied to the representative value of s_u for undrained conditions and to $\tan\phi'$ for drained conditions (not to ϕ').

A.7.7.1.3 Yield surfaces for selected cases

Historically, yield surfaces for undrained conditions have been based on analytical and numerical studies while those for drained conditions have been derived from experimental studies. The latter is due to the relative complexity of an analytical approach for drained soil conditions.

A.7.7.1.4 Undrained conditions

A.7.7.1.4.1 Surface foundations with zero-uplift capacity along the foundation–soil interface

The general 'scallop-shaped' form of the three-dimensional yield surface in vertical, horizontal and moment loading space for undrained failure of a surface foundation with zero-uplift resistance along the foundation–soil interface is shown in Figure A.8 in terms of normalised actions, Q/Q_{ult}, H/H_{ult} and M/M_{ult} (vertical action Q is denoted by V in Figure A.8). The surface is symmetrical in the H–M plane and exhibits diminishing moment capacity as vertical action (Q) falls below 0,5 Q_{ult} as the foundation begins to lift-off from the seafloor.

References [321] and [144] derive yield surfaces for circular and rectangular surface foundations for uniform undrained soil strength with depth and the latter presents approximating expressions for the shape of the yield surface and the uniaxial capacities defining its apex points. Further studies have presented approximating expressions for yield surfaces capturing the effect of linearly increasing undrained strength with depth and loading in six degrees of freedom (VH2M2T), [306],[315].

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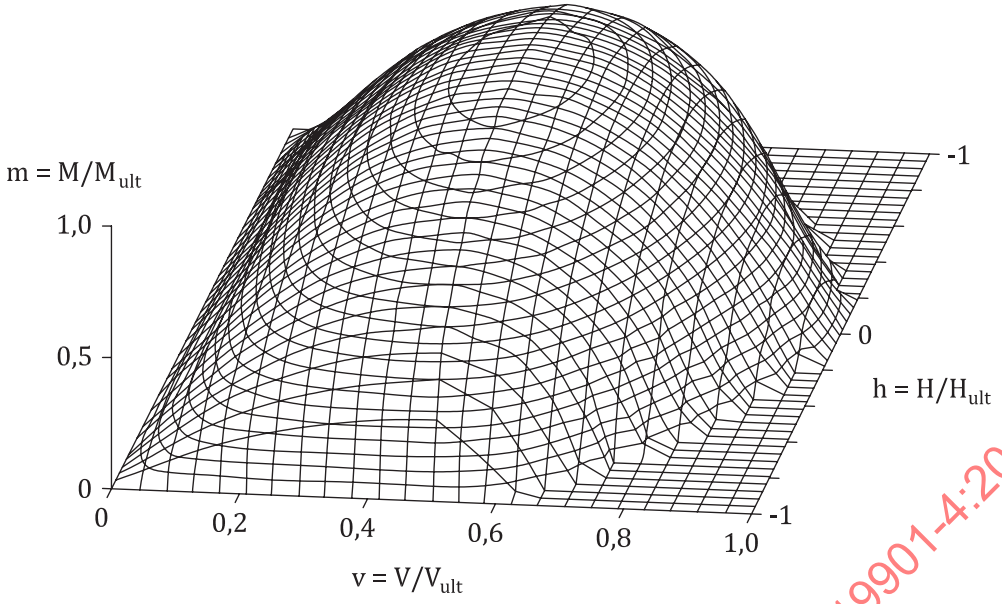


Figure A.8 — Yield surface for undrained conditions for a surface foundation with zero-uplift capacity along the foundation–soil interface [321]

A.7.7.1.4.2 Surface foundations with unlimited-uplift capacity along the foundation–soil interface

In some cases, uplift resistance can be mobilised beneath surface or skirted foundations due to suction and can be relied on for the duration over which undrained conditions prevail.

Uplift resistance provided by foundation skirts can be represented by modelling a surface foundation with an unlimited tension interface. The general form of the yield surface for undrained failure of a surface foundation with an unlimited tension interface is shown in Figure A.9 (vertical action Q is denoted by V in Figure A.9). The 'walnut-shaped' surface is asymmetric in the H–M plane, with maximum moment capacity mobilised in conjunction with a horizontal action acting in the same direction (i.e. clockwise and left-to-right or vice versa). Moment capacity continues to increase with diminishing vertical action, contrary to the zero-tension interface case, as a foundation with unlimited-uplift capacity will not lift-off from the seafloor.

Yield surfaces and accompanying approximating expressions have been derived for strip, rectangular and circular foundations, both surface and embedded, homogeneous and heterogeneous undrained soil strength profiles, strength profiles with a surficial crust, and consolidated shear strength, although not comprehensively for all combinations (e.g. References [47], [142], [143], [121], [122] and [307]).

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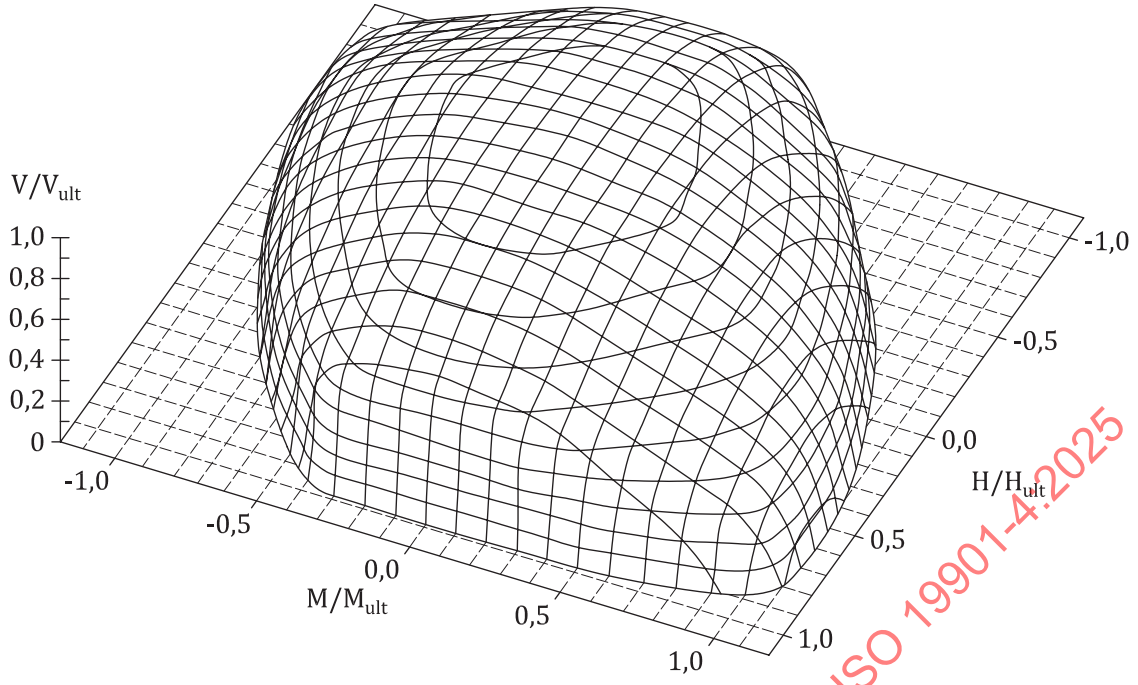


Figure A.9 — Yield surface for undrained conditions for a surface foundation with unlimited-uplift capacity along the foundation-soil interface [48]

The shape of the yield surface for foundations with unlimited-uplift capacity depends on foundation geometry and soil strength profile. In some cases, the normalized size of the yield surface decreases with increasing degree of soil strength heterogeneity; therefore scaling a yield surface derived for homogeneous soil strength by uniaxial ultimate limit states appropriate to a heterogeneous soil strength profile will be un-conservative [147], [144].

A.7.7.1.4.3 Embedded foundations

The coupling of the horizontal and moment degrees of freedom when a foundation is physically embedded leads to an asymmetric and oblique failure surface in the *H–M* plane. The asymmetry and obliqueness become more pronounced with increasing embedment ratio. The general form of a yield surface for undrained failure of embedded foundations is shown in Figure A.10.

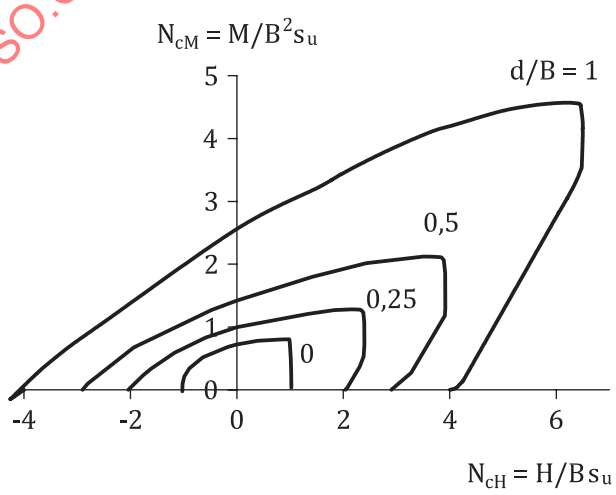


Figure A.10 — Change in shape of yield surface for undrained conditions with foundation embedment d/B [151]

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References [142] and [123] present yield surfaces in general loading space for strip and rectangular shallow foundations with embedment ratios in the range zero to one for uniform soil strength and linearly increasing strength with depth.

Existing studies have generally considered embedment in terms of a solid plug, although the capacity of a skirted foundation can be reduced because of the intrusion of the failure mechanism into the soil plug [49]. Modelling a skirted foundation as a solid plug is based on the assumption that sufficient internal skirts are provided to ensure the soil plug displaces as a rigid body. Numerical studies for assessment of critical skirt spacing are presented in References [1] and [120].

A.7.7.1.5 Drained conditions

A.7.7.1.5.1 General

Yield surfaces for drained conditions incorporate isotropic strain-hardening to accommodate increasing shear strength with increasing stress level. The shape of the yield surface is assumed to be unique and the isotropic expansion and contraction of the surface is defined by a hardening rule (vertical resistance–displacement relationship). Under drained conditions tension cannot be sustained beneath a foundation and therefore the foundation will lift-off from the seabed under moment loading in conjunction with vertical actions, typically for $Q < 0{,}5\ Q_{ult}$ (see References [60] and [241]).

A.7.7.1.5.2 Surface foundations

The general form of the yield surface for drained failure of a surface foundation is shown in Figure A.11. The 'rugby ball-shaped' surface is parabolic in planes of *QH* and *QM* and a rotated ellipse in the *HM* plane (vertical action, *Q*, is denoted by *V* in Figure A.11). Maximum horizontal action and moment capacity are mobilised in conjunction with a vertical action $Q = 0{,}5\ Q_{ult}$ and maximum moment capacity is mobilised in conjunction with horizontal action acting in opposition (i.e. clockwise and right-to-left or vice versa).

Reference [60] proposed the yield surface shown in Figure A.9 along with a closed-form expression to describe its shape. The yield surface was based on results from various experimental studies on rough, rigid, plane strain and rectangular shallow foundations on dense silica sand. A subsequent study considering circular foundations on loose carbonate sand showed a similar form of yield surface and for which a closed-form expression was proposed [61].

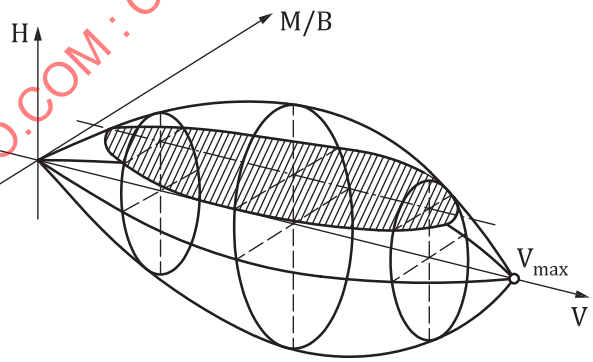


Figure A.11 — Yield surface for drained conditions for a surface foundation [60]

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A.7.7.1.5.3 Embedded foundations

The additional capacity available from foundation embedment is accounted for by scaling the envelope for a surface foundation (as shown in Figure A.11) by its apex points H_{max}/Q and M_{max}/Q [241], [150].

A.7.7.2 Risk-informed decision making

A design approach following risk-informed decision making can be applied on a case-by-case basis, through system-based design ensuring that the integrity, safety and operability of the system are not compromised,

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and consequences of failure and likely failure mechanisms are robustly quantified, considering the accumulation of foundation displacements and rotations due to repeated loading.

System failure modes, due to short-term or long-term events, that might require evaluation in a risk-informed design of subsea systmes, can include:

a) validating surface sliding of foundations over the seafloor i.e. ensuring bearing failure of the seafloor is prevented;

b) over-straining/overstressing of pipes, spools/jumpers and connected flowlines;

c) problems with the installation of connections (e.g. pitch rotation beyond the installation tolerance prior to tie-in operations);

d) not fulfilling the specified function or performance standard (e.g. a riser hold-back anchor being pulled-out and no longer supporting the riser; excessive sliding of buckle initiator).

The design of foundations where foundation movement over the seafloor is permitted is discussed in References [81], [88] and [97].

A.7.8 Installation

A.7.8.1 General

When installing shallow foundations, the vertical heave motions induced by the vessel can cause the touchdown to be an impact between the seafloor and the structure. This impact is normally controlled by limiting weather criteria and installation operations. Foundation failure during installation, mainly in soft soil conditions, can be avoided if the installation is performed by use of a heave compensator and low rate of descent towards the seafloor (<0,2 m/s).

Vessel heave motions greater than crane pay-out speed can result in the structure having excessive velocity at impact with the seafloor and multiple set-downs. As a result, the foundation can be pulled up after first set-down, thus generating a pull-out failure in the soil (reverse bearing capacity failure).

A.7.8.2 Skirt penetration resistance

The skirt resistance to penetration can be estimated following formulae in A.11.5.2.2.1.

The end bearing components can be estimated by bearing capacity formulae or alternatively by the direct use of cone penetrometer resistance q_c corrected for shape difference and overburden. It is possible that the latter is not directly applicable for wide concrete skirts. The shaft friction resistance can be determined by laboratory testing or other suitable experience.

Previous relevant installation experience should be considered (see Reference [82]).

The skirt penetration rate should be such that removal of the water trapped within the skirt compartments can be accomplished without forcing it under the skirts and damaging the foundation.

An uneven seafloor, lateral soil strength variability, existence of boulders, drill cuttings and cement can cause uneven penetration or structural damage of skirts. In some cases, site improvements are used such as levelling the area by dredging or fill emplacement."

A.7.8.3 Required and allowable under-pressure

In general, analysis is more straightforward in lower permeability soils (e.g. clays).

In high permeability soils (e.g. sands) water flow is induced within the soil, which increases the volume of water to be removed from the skirted foundation and which alters the effective stress distribution in the soil.

Further guidance is provided in A.11.5.2.2.1.

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A.7.9 Relocation, retrieval and removal

During removal or retrieval of a skirted shallow foundation, suction forces will tend to develop at the foundation base and the tips of skirts. These forces can usually be overcome by sustained uplift forces (up to the submerged weight of the foundation) and by introducing water into the base compartments to relieve the suction.

Set-up effects can result in higher extraction resistance than installation resistance.

In the case of un-skirted foundations or shallow foundations with short skirts, it is often preferable to lift initially from one corner or side of the foundation, to introduce drainage, and reduce suction effects.

A.8 Guidance on pile foundation design

A.8.1 Pile capacity for axial compression

A.8.1.1 General

No additional guidance is offered.

A.8.1.2 Axial pile capacity

In conventional static capacity-based design, the pile design actions (factored permanent and variable actions plus factored extreme environmental actions) are compared against the factored pile capacity. The factored actions are defined in ISO 19902. The pile capacity is defined as the integrated friction and tip resistance (see 8.1 and 8.2).

It is not always correct to add the representative value of the end bearing to the representative value of the skin friction to obtain the representative value of the axial capacity of a pile. This subject is addressed in References [243], [276] and [328].

A.8.1.3 Skin friction and end bearing in clay soils

A.8.1.3.1 General

Estimating pile capacity in clay soils requires judgment in selecting design parameters and in interpreting calculated capacities. Some of the items that should receive design consideration are detailed in A.8.1.3.2.

A.8.1.3.2 Axial pile capacity in clay

A.8.1.3.2.1 Loading test database for piles in clay

Further information on the load test database for piles in clay can be found in References [302] and [210].

A.8.1.3.2.2 Unified CPT method for pile axial capacity in clays

This subclause presents the 'unified CPT method' for assessing driven pile capacity in clays [209]. Formulae (A.38) and (A.39) are recommended for the evaluation of the static capacity of steel piles after consolidation induced by pile driving is completed. Potential long-term ageing effects are not included. The method can be applied to clays in yones 2, 3 and 4 on the soil behaviour type (SBT) chart as shown in Figure A.12, Assessment of pile capacity for sensitive clays that fall within SBT chart Zone 1 should include local experience, pile testing and alternative design approaches.

The reliability of the method has been evaluated and the method was shown to provide pile foundations that are more reliable than those obtained using the main text method in 8.1.3. The unified CPT method should only be applied to driven piles and should not be applied to vibro-driven piles unless the vibrated portion of the pile is restricted to the initial 20 % of pile penetration. The method is developed for offshore steel piles without paint, coatings or mill-scale varnish that are likely to reduce pile roughness below that

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usually expected (typically around 10 microns centre line average roughness for steel used for offshore piles), and negatively affect the interface friction that can be mobilised. Further information on the method and Formulae (A.38) and (A.39) is provided in Reference [209].

The cone resistance, q_t, includes a resistance component due to hydrostatic water pressure which does not affect unit skin friction. However, the formulations were derived by correlating f(z) with q_t rather than $q'_t = q_t - u_0$, where u_0 is the hydrostatic pore pressure (below seabed level where q_t is zeroed). This simplification had no discernible impact on the degree of fit obtained against the database of load tests used for calibration of Equation (A.38).

For the unified CPT method in clays, the (fully consolidated) peak unit skin friction for capacity, $f(z)$, in stress units, at depth, z, can be calculated using Formula (A.38):

$$f(z) = 0{,}07 F_{st} q_t \left[\max\left(\frac{h}{D^*}, 1 \right) \right]^{-0{,}25} \quad \text{(A.38)}$$

where

F_{st} is 1 for clays with $I_{z1} > 0$, in Zones, 2, 3 and 4 on the SBT chart;

F_{st} is 0,5 ± 0,2 clays with $I_{z1} < 0$, in Zone 1 on the SBT chart;

q_t is the corrected CPT cone resistance as defined in ISO 19901-8:2023, 8.3.

h is the distance above the pile tip at which $f(z)$ acts (= $L-z$)

L is pile embedment length;

$D^* = \left(D^2 - D_i^2\right)^{0,5}$ for an open-ended pile and $D^* = D$ for a closed-ended pile;

D is the pile outer diameter of a pipe pile;

D_i is the internal diameter of a pipe pile

I_{z1} can be calculated by:

$$I_{z1} = Q_{tn} - 12\exp^{(-1{,}4 F_r)}$$

where F_r is the normalized friction given by $F_r = f_s / (q_t - \sigma_{v0}) \times 100\ \%$, in which f_s is the CPT sleeve friction and σ_{v0} is the total vertical stress below the seafloor level.

t-z curves can be determined using the friction value combined with the *t-z* curve definition for clays in 8.4.1.

In the absence of more definitive criteria, the unit end bearing resistance applied across the full base area of a large diameter pile with a length to diameter ratio greater than five can be calculated using Formula (A.39):

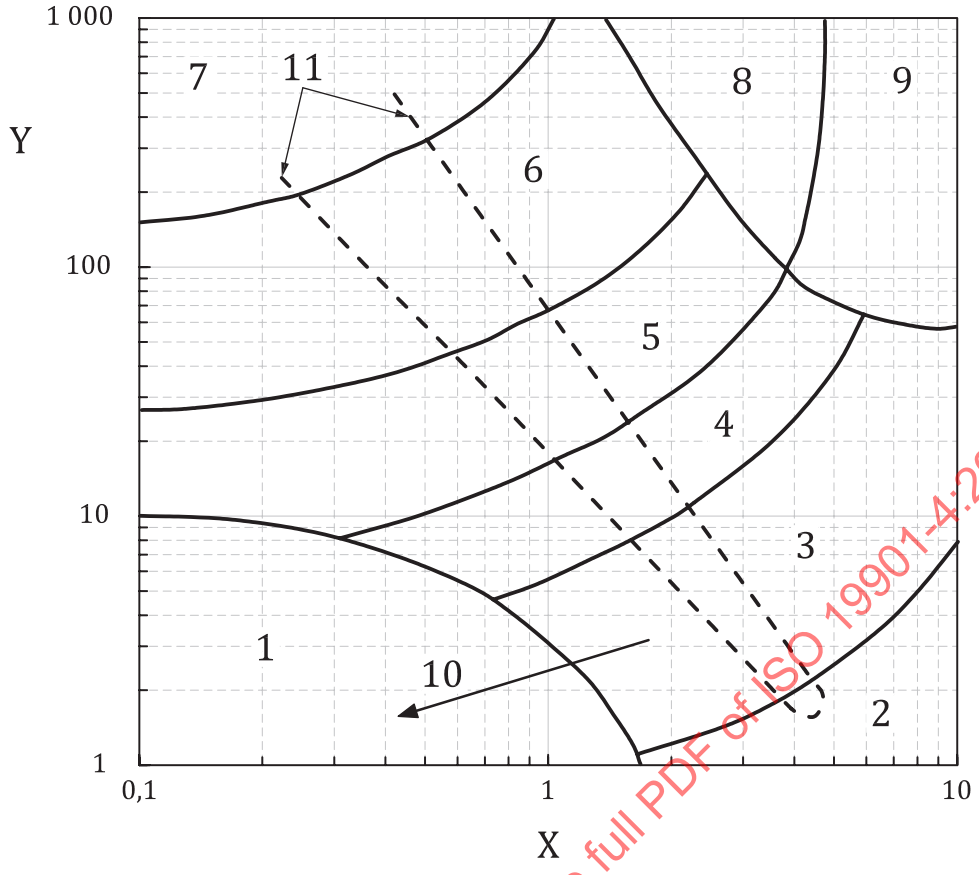
$$q_{b0,1} = q_t \left[0{,}2 + 0{,}6 \left(\frac{D^*}{D} \right)^2 \right] \quad \text{(A.39)}$$

Where q_t is the average corrected CPT end resistance within $20t$ below the pile tip, in which t is the pile wall thickness. The unit end bearing should be taken as plugged resistance acting across the entire cross-section and no additional allowance should be made for internal skin friction.

If used for foundations with a length to diameter ratio less than five, Formulae (A.39) should be validated.

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Key

X	normalized friction ratio, Fr	6	zone 6 – sands; clean sand to silty sand
Y	normalized cone resistance, Qtn	7	zone 7 - gravelly sand to sand
1	zone 1 – sensitive clays and silts	8	zone 8 - very stiff sand to clayey sand
2	zone 2 – organic soils, peats	9	zone 9 - very stiff fine grained
3	zone 3 – clays – clay to silty clay	10	increasing sensitivity
4	zone 4 – silt mixtures: clayey silt to silty clay	11	zone of normally consolidated soil
5	zone 5 – sand mixtures -silty sand to sandy silt		

Figure A.12 — Soil behaviour type (SBT) chart [291]

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A.8.1.3.2.3 Establishing representative strength and effective overburden stress profiles

The axial pile capacity in clay is directly influenced by the undrained shear strength and effective overburden stress profiles selected for use in analyses. The wide variety of sampling techniques and laboratory and in situ testing techniques and the scatter in the shear strength data from the various types of tests complicate selection of representative shear strength profiles. ISO 19901-8 provides additional information on the sampling and laboratory testing techniques and on the quality of marine soil investigations.

Strength profiles are typically established by combining the results of laboratory tests on undrained unconsolidated samples (e.g. UU triaxial or minivane tests), laboratory tests on consolidated samples (e.g. direct simple shear tests or consolidated triaxial tests in compression or extension), and in situ tests (e.g. cone, ball, or T-bar penetration tests, in situ vane tests). In soft and very soft clays, some test (e.g. pocket penetrometer, torvane, fall cone) can be unreliable.

In selecting the representative shear strength value, the sampling and testing techniques used to correlate the shear strength to any available relevant pile loading test data should be considered. The experience with pile performance can also play an important role in assessing the appropriate representative shear strength value.

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In situ testing with a vane or penetrometers, such as the cone, piezocone, ball and T-bar, will help in assessing sampling disturbance effects and can provide a continuous shear strength profile. Approaches. such as the SHANSEP (stress history and normalized soil engineering properties) technique as described in References [200] and [201] can help providing a more consistent interpretation of the shear strength profile.

A.8.1.3.2.4 Pile length effect

Long piles driven in clay soils are typically axial flexible and can therefore experience capacity degradation due to progressive failure in the soil due to strength reduction (strain softening) with continued displacement or shearing of a particular soil horizon during pile installation;

The occurrence of degradation due to these effects depends on many factors related to both installation conditions and soil behaviour. Methods of estimating the possible magnitude of reduction in capacity of long piles can be found in Reference [279].

A.8.1.3.2.5 Changes in axial capacity in clay with time

Existing axial pile capacity calculation procedures for piles in clay are based on experience assisted by the results of axial pile loading tests. In these tests, few of the piles were instrumented and in most cases little or no consideration was given to the effects of time after driving on the development of pile-soil shear resistance. Axial capacity of a driven pipe pile in clay computed in accordance with 8.1.2 and 8.1.3 is intended to represent the long-term static capacity of piles in undrained conditions when subjected to axial actions until failure after dissipation of excess pore water pressure caused by the installation process. Immediately after pile driving, pile capacity in a clay deposit can be significantly lower than the ultimate static capacity. Field measurements have shown that the time required for driven piles to reach ultimate capacity in a clay deposit can be relatively long, as much as two to three years [75][42][206]. However, the rate of strength gain is highest immediately after driving, and this rate decreases during the dissipation process. Thus, a significant strength increase can occur in a relatively short time.

During pile driving in normally consolidated to lightly over-consolidated clays, the soil surrounding a pile is significantly disturbed, the stress state is altered, and large excess pore pressures can be generated. After installation, these excess pore pressures begin to dissipate, i.e. the surrounding soil mass begins to consolidate and the pile capacity increases with time. This process is usually referred to as set-up. The rate of excess pore pressure dissipation is a function of the coefficient of radial (horizontal) consolidation, pile radius, plug characteristics (plugged versus unplugged pile), and soil layering.

In the case of driven pipe piles supporting a structure where the design actions can be applied to the piles shortly after installation, the time-consolidation characteristics should be considered in pile design. In such cases, the capacity of piles immediately after driving and the expected increase in capacity with time are important design variables that can impact the safety of the foundation system during early stages of the consolidation process.

A number of investigators have proposed analytical models of pore pressure generation and the subsequent dissipation process for piles in normal to lightly over-consolidated clays [239] [282][358]. Since excess pore pressures are generated by pile driving operations, any dissipation of the excess pore pressures after installation should correspond to an increase in the shear strength of the surrounding soil mass and hence an increase in the capacity of the pile. After dissipation of excess pore pressures, the capacity of a pile approaches long-term capacity, although some strength gain can continue due to secondary processes. In some over-consolidated clays, pile capacity can decrease as pore pressures dissipate, provided the rate of change of radial total stress decreases faster than the rate of change of pore pressure. The analytical models account for the degree of plugging by assuming various degrees of plug formation, ranging from closed- to open-ended pile penetration modes. Input necessary for the analysis includes the soil characteristics (compressibility, stress history, strength, etc.) and the initial site conditions.

This sub-clause does not necessarily apply to the evaluation of set-up for soils with different plasticity characteristics and under different states of consolidation (especially over-consolidated clays) and piles with D/WT (pile outer diameter/pile wall thickness) ratios greater than 40.

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A.8.1.4 Skin friction and end bearing in sands

A.8.1.4.1 General

This subclause provides guidance for sands that do not fit within the constraints presented in 8.1.4. In particular for coarse sands, normal effective stress levels outside of those considered within the pile database, higher fines content or for unconventional mineralogy, it can be appropriate to consider further laboratory testing including special ring shear interface friction tests [177] and to use an alternative pile design method, such as the CPT method presented in 8.1.4. The method presented in 8.1.4 was calibrated based upon pile tests where the soil generally had a soil behaviour type index, I_c, larger than 2,1, i.e. they were in the category of sand mixtures (silty sand to sandy silt) using the soil behaviour type chart of Figure A.12.

Recent studies (see Reference [162]) involving ring-shear tests against steel interfaces of appropriate roughness that were designed to replicate driven pile installation suggest that the large shear displacements implicit in pile driving lead to a modest dependency of ultimate (design) δ_{cv} on mean grain size D_{50} among standard pure silica test sands. Reference [216] used the same tools to investigate the influence of non-plastic silt and non-plastic fines contents (up to around 20 %) as well as stress level dependency for natural sand samples taken at both onshore and offshore sites, finding that both variables can affect ultimate critical state (design) interface friction angles, with the latter showing modest increases as normal stress levels rise due to greater interaction of the grains with the steel surface. In cases where such factors merit investigation, site-specific tests that account for pile installation effects can be conducted following the procedures set out in [177].

A.8.1.4.2 Parameter value assessment

The soil investigation should provide information that is adequate to capture the spatial variability, horizontally and vertically, of the boundaries and parameter values of all layers.

For any CPT-based method, the computed pile capacity in sand is most sensitive to cone penetration resistance, q_c, followed by $\tan\delta_{cv}$ and σ'_{v0}. Since an accurate capacity assessment is a function of the accuracy of both the model and the parameters, guidance regarding selecting appropriate parameter values is given in items a) to c).

a) Parameter, $q_c(z)$

The CPT should measure $q_c(z)$ with apparatus and procedures that are in general accordance with ISO 19901-8. In particular, ISO 19901-8 prescribes cones with a base area in the range of 500 mm^2 to 2 000 mm^2 and a penetration rate of 20 mm/s ± 5 mm/s.

The CPT-based design methods were established for cone resistance values, q_c, up to 100 MPa. The use of the method should be validated when applying to sands and sandy tills with higher resistances.

A measured, continuous profile of $q_c(z)$ is preferable to an assumed/interpolated discontinuous profile, but is generally not achievable offshore at large depths below the seafloor with a down-hole CPT apparatus. This is generally due to factors such as limited stroke and/or maximum resistance being achieved. When (near) continuous $q_c(z)$ profiles are needed, overlapping CPT push strokes can be considered.

With discontinuous CPT data, a 'blocked' $q_c(z)$ profile can be used, where the soil profile is divided into layers, in each of which $q_c(z)$ is assumed to vary linearly with depth. 'Blocked' profiles should be assessed, particularly when they contain maximum q_c values at the ends of CPT push strokes. When the push strokes contain no maximum $q_c(z)$ data, a moving window can be used to determine the average profile (and its standard deviation), through which a straight line can be fitted. If present, thin layers of weaker material (e.g. silt or clay) should be modelled.

For geotechnical investigations where several vertical CPT profiles have been made (e.g. one per platform leg), it is suggested that at least two approaches be employed: pile capacity should first be based on the combined averaged $q_c(z)$ profile and then based on individual $q_c(z)$ profiles. Judgment is required to select the most appropriate $q_c(z)$ profile and to determine the associated final axial capacity.

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b) Parameter, $\sigma'_{v0(z)}$

Usually, pore water pressures in sands are hydrostatic and in this case $\sigma'_{v0(z)}$ equals ($\gamma' z$), where γ' is the submerged soil unit weight. Offshore sands are generally very dense and often silty. In general, design γ' values in sands should be based on measured laboratory values (corrected for sampling disturbance effects), which should be compatible with relative density, D_r, estimated from $q_c(z)$ and maximum and minimum dry unit weight values determined in the laboratory.

c) Scour

Scour (seabed erosion due to wave and current action) can occur around offshore piles. Common types of scour are general scour (overall seabed erosion) and local scour (steep-sided scour pits around single piles or pile groups). There is no generally accepted method to account for scour in axial capacity for offshore piles. Reference [346] [60] [245] [327] give techniques for scour depth assessment. In addition, general scour data can be obtained from national authorities.

In lieu of project specific data, guidance on local scour depth is provided in A.8.5.3.

Scour decreases axial pile capacity in sand. Both friction and end bearing components are usually affected. This is because scour reduces both $q_c(z)$ and σ'_v (vertical effective stress). For excavations (i.e. general scour), Reference [250] recommends that $q_c(z)$ is simply proportional to $\sigma'_{v0}(z)$, i.e.:

$$q_{c,f}(z) = \chi\, q_{c,0}(z) \tag{A.40}$$

where

$q_{c,f}(z)$ is the final reduced CPT cone-tip resistance at depth z, after general scour (in stress units);

$q_{c,0}(z)$ is the original CPT cone-tip resistance at depth z, before general scour (in stress units);

χ is the dimensionless scour reduction factor ($\chi = \sigma'_{vf}/\sigma'_{v0}$);

σ'_{vf} is the final vertical effective stress value, after scour (in stress units);

σ'_{v0} is the original vertical effective stress value, before scour (in stress units).

For large general scour depths and normally consolidated sands, an alternative and conservative approach as described in Reference [129] can be used to determine χ from:

$$\chi = \frac{1}{1+2K_0}\sqrt{\frac{z'+2K_0\sqrt{\Delta z_{GS}\times z'+z'^2}}{\Delta z_{GS}+z'}} \tag{A.41}$$

where

Δz_{GS} is the general scour depth (in m);

z' is the final depth below seafloor, after general scour, ($z' = z - \Delta z_{GS}$) (in m);

K_0 is the coefficient of lateral earth pressure at rest, the ratio of the effective horizontal to vertical in situ soil stresses, $K_0 = \sigma'_{h0}(z)/\sigma'_{v0}(z)$.

A method to reduce the effective stress, σ'_v, for both general and local scour is provided in A.8.5.

A.8.1.4.3 Former main text method in sands

The former main text method in sands is presented in this subclause due to its historical use for many previous pile designs, although it should no longer be used for pile design or assessment of pile capacity in sand. In exceptional cases, it can be used for assessment of pile capacity for existing piles where there are no CPT data available to use a CPT-based method. In comparison to the method described in this subclause,

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the unified CPT-based method presented in 8.1.4 is considered to be fundamentally better and has shown statistically closer predictions of pile loading test results and is the preferred method.

With the former main text method, for driven pipe piles in sands, the unit skin friction, $f(z)$, in stress units, at depth, z, can be calculated by:

$$f(z) = \beta\sigma'_{vo(z)} \quad \text{(A.42)}$$

where

β is the dimensionless skin friction factor, for sands;

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

In the absence of specific data, β values for open-ended pipe piles that are driven unplugged can be taken from Table A.3. For full displacement piles (i.e. closed-ended or fully plugged open-ended piles), values of β can be assumed to be 25 % higher than those given in Table A.3. For long piles, $f(z)$ does not necessarily increase linearly with the overburden stress as implied by Formula (A.42). In such cases, it is appropriate to limit f to the values given in Table A.3.

For end bearing of piles in sands, the unit end bearing, q, in stress units, can be computed using Formula (A.43):

$$q = N_q\sigma'_{vo,tip} \quad \text{(A.43)}$$

where

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

N_q is the dimensionless bearing capacity factor.

Recommended N_q values are presented in Table A.3. For long piles, q does not necessarily increase linearly with the overburden stress as implied by Formula (A.43). In such cases, it is appropriate to limit q to the values given in Table A.3. For plugged piles, the bearing pressure can be assumed to act over the entire cross-section of the pile. For unplugged piles, the bearing pressure acts on the pile annulus only. In this case, additional resistance is offered by friction between the soil plug and the inner pile wall. Whether a pile is considered to be plugged or unplugged should be based on static calculations using a unit skin friction on the soil plug equal to the outer skin friction. A pile can be driven in an unplugged condition, but can behave as plugged under static actions.

For soils that do not fall within the ranges of relative density and soil description given in Table A.3, or for materials with unusually weak grains or compressible structure, Table A.3 is not necessarily appropriate for selection of design parameters. For example, unconventional soils such as very loose soils or soils containing large amounts of mica or volcanic grains can require special laboratory or field tests for selection of design parameters. Sands containing calcium carbonate, which are found extensively in many areas of the oceans (see A.6.3), are of particular importance.

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

In layered soils, skin friction values, $f(z)$, in sand layers should be computed in accordance with Table A.3. End bearing values for piles tipped in sand layers can also be taken from Table A.3.

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Table A.3 — Design parameters for siliceous sand

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

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

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

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

[b] The skin friction factor β (equivalent to the '$K \tan\delta$' term used in the past) is introduced in this document to avoid confusion with the δ parameter used in the past.

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

[d] Design parameters proposed in the past for these relative density/soil description combinations can be unconservative. Hence, the unified CPT method should be used for these soils (see 8.1.4).

A.8.1.5 Skin friction and end bearing in gravels

No additional guidance is offered.

A.8.1.6 Skin friction and end bearing of grouted piles in rock

No additional guidance is offered.

A.8.1.7 Skin friction and end bearing of driven piles in intermediate soils

No additional guidance is offered.

A.8.2 Pile capacity for axial tension

No additional guidance is offered.

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A.8.3 Axial pile performance

A.8.3.1 Static axial behaviour of piles

No additional guidance is offered.

A.8.3.2 Cyclic axial behaviour of piles

A.8.3.2.1 Qualification

Modelling cyclic effects explicitly can improve the designer's insight into the relative importance of the cyclic characteristics of the actions. Historically, cyclic effects have been taken into account implicitly rather than explicitly. Design methods developed and calibrated on an implicit basis generally need extensive modification where explicit algorithms are employed.

A.8.3.2.2 Actions

Axial actions on piles are developed from a wide variety of operating, structural and environmental sources. Permanent and variable actions are generally long duration actions and are often referred to as static actions. Environmental actions are developed by winds, waves and currents, earthquakes and ice floes. These actions can have both low and high frequency cyclic components in which the rates of change of actions and action durations are measured in seconds. Storm and ice can cause several thousand cycles of (relatively speaking) low frequency actions, while earthquakes can induce several tens of cycles of high frequency actions.

A.8.3.2.3 Cyclic effects

Cyclic effects should be considered when there are unusual limitations on pile penetrations or when certain soils, conditions related to actions or novel structures (e.g. compliant towers) are involved.

Compared with long-term static actions, cyclic actions can have the following important influence on pile axial capacity and stiffness:

a) decrease capacity and stiffness due to repeated actions [51];

b) increase capacity and stiffness due to high rates of change of actions [52].

The resultant effect on capacity is primarily influenced by the pile properties (stiffness, length, diameter, material), the soil characteristics (type, stress history, strain rate and cyclic degradation) and the action characteristics (numbers and magnitudes of repeated actions). Cyclic actions can also cause accumulation of pile displacements and either stiffening and strengthening or softening and weakening of the soils around the pile. Hysteretic and radiation damping dissipate the energy provided by the actions in the soil. For earthquakes, the free-field ground motions (independent of the presence of the piles and structure) can develop important cyclic straining effects in the soils. These effects can influence pile capacity and stiffness.

Additional guidance on the effect of cyclic actions on pile axial capacity and stiffness can be found in Reference [178].

A.8.3.2.4 Analytical models

A.8.3.2.4.1 General

A variety of analytical models have been developed and applied to determine the cyclic axial behaviour of piles. These models can be grouped into two general categories, discrete element models and continuum models.

A.8.3.2.4.2 Discrete element models

The soil around the pile is idealized as a series of uncoupled 'springs' or elements attached between the pile and the far field soil (usually assumed rigid). The material behaviour of these elements can vary from

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linearly elastic to nonlinear, hysteretic and rate dependent. The soil elements are commonly referred to as *t*–*z* (friction resistance–displacement) and *Q*–*z* (tip resistance–displacement) elements (see References [231], [266], [36] and [191]). Linear or nonlinear dashpots (velocity dependent resistances) can be placed in parallel and in series with the discrete elements to model radiation damping and rate of change of loading effects [36]. The pile can also be modelled as a series of discrete elements (e.g. rigid masses interconnected by springs), or modelled as a continuous rod, either linear or nonlinear. In these models, material properties (soil and pile) can vary along the pile.

A.8.3.2.4.3 Continuum models

The soil around the pile is idealized as a continuum attached continuously to the pile. The material behaviour can incorporate varying degrees of nonlinearity and heterogeneity in its stress-strain rule. Again, the pile is typically modelled as a continuous rod, either linear or nonlinear. In these models, material properties can vary in any direction [253], [292].

Once the idealized model is established and the relevant formulae are developed, a solution technique should be selected. For simple models, a closed-form analytical approach is sometimes possible. Otherwise, a numerical procedure should be used. In some cases, a combination of numerical and analytical approaches is helpful. The most frequently used numerical solution techniques are the finite difference method and the finite element method. Either approach can be applied to both the discrete element and continuum element models. Discrete element and continuum element models are occasionally combined [266]. Classical finite element models have been used for specialized analyses of piles subjected to monotonic axial actions [253].

Discrete element models, solved numerically, have seen the most use in evaluation of piles subjected to cyclic action. Results from these models are used to develop information on pile accumulated displacements and on pile capacity following high intensity cyclic actions [266], [191].

Elastic continuum models solved analytically (similar to those used in machine vibration analyses) have proven to be useful for evaluations of piles subjected to low intensity, high frequency cyclic actions at or below design working levels [253], [292]. At higher intensity actions, where material behaviour is likely to be nonlinear, the continuum model solved analytically can still be used by employing equivalent linear properties that approximate the nonlinear, hysteretic effects [222].

A.8.3.2.5 Soil characterisation

A key part of developing analytical models to evaluate cyclic effects on piles is the characterization of soil-pile interaction behaviour with in situ, laboratory and model-prototype pile loading tests. In developing soil characterizations, pile installation and conditions of the actions on a pile are integrated into the testing programmes [191].

In situ tests (e.g. vane shear, cone penetrometer, ball or T-bar penetrometer) can provide insights into in-place soil behaviour and stress-strain properties [221], [234]. Both low and high amplitude stress-strain properties can be developed. Long-term (static, creep), short-term (dynamic, impulsive) and cyclic (repeated) actions can be simulated with in situ testing equipment.

Laboratory tests on representative soil samples permit a variety of stress-strain conditions to be simulated and evaluated [350]. Soil samples can be modified to simulate pile installation effects (e.g. remoulding and reconsolidating to estimated in situ stresses). The samples can be subjected to different boundary conditions (triaxial, simple-shear, interface shear) and to different levels of sustained and cyclic shear time histories to simulate in-place conditions of applied actions.

Tests on model and prototype piles are another source of data for developing soil characterizations for cyclic analyses. Model piles can be instrumented and repeated tests can be performed in soils and for a variety of actions [191], [1]. Geometrical scale, time scale and other modelling effects should be addressed in applying results from model tests to analyses of prototype behaviour.

Data from loading tests on prototype piles are useful for calibrating analytical models [262], [236], [133], [260]. Such tests, even if not instrumented, can provide data to guide development of analytical models. These tests can also provide data for verifying results of soil characterizations and analytical models, as shown in [191], [36], [263] [37]. Prototype pile loading tests coupled with in situ and laboratory soil testing and analytical

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models can provide a framework for making realistic evaluations of the responses of piles to cyclic axial actions.

A.8.3.2.6 Analysis procedure

A.8.3.2.6.1 Actions

The actions on the pile head should be characterized in terms of their magnitudes, durations, sequence and numbers of cycles. This includes both long-term actions and short-term cyclic actions. Typically, the design static and cyclic actions expected during a design event are chosen.

A.8.3.2.6.2 Pile properties

The properties of the pile including its diameter, wall thickness, stiffness, weight and length should be defined. This will require an initial estimate of the pile penetration that is appropriate for the design actions. Empirical, pseudo-static methods based on pile loading tests or soil tests can be used to make such estimates.

A.8.3.2.6.3 Soil properties

Different analytical approaches will require different soil parameters. For the continuum model, the elastic and damping properties of the soil are required. In the discrete element model, soil resistance–displacement relationships along the pile shaft (t–z) and at its tip (Q–z) should be determined. *In situ* and laboratory soil tests and model and prototype pile loading tests can provide a basis for such determinations. These tests should at least implicitly include the effects of pile installation, types of actions and time scales. In addition, the test should be performed so as to provide insight regarding the effects of the characteristics of the actions on the pile. Most importantly, the soil behaviour characteristics should be appropriate for the analytical model(s) used, duly recognizing the empirical bases of these models.

A.8.3.2.6.4 Cyclic analyses

Analyses should be performed to determine the response (resistance and displacement) characteristics of the pile subjected to its design static and cyclic actions. Recognizing the inherent uncertainties in evaluations of pile actions and soil-pile behaviour, parametric analyses should be performed to evaluate the sensitivity of the pile response to these uncertainties. The analytical results should develop realistic predictions of pile resistance and accumulated displacements for design actions. In addition, following the simulation of static and cyclic design actions, the pile should be further analysed so as to estimate its reserve capacity.

A.8.4 Soil reaction for piles under axial compression

A.8.4.1 Axial shear transfer t–z curves

Theoretical curves can be constructed in accordance with Reference [196]. Empirical t–z curves based on the results of model- and full-scale pile loading tests can follow the procedures for clay soils described in Reference [90].

The representative pile capacity model in 8.1.2 does not provide any information about axial pile displacements, which are important for serviceability limit states, especially in non-extreme conditions for actions due to permanent, variable and operating environmental actions that are generally well below the design actions. In cases where the representative axial capacity of 8.1.2 is adopted, the axial shear transfer characteristics between pile and soil can be derived as described in 8.4, and analytical models can be employed to investigate axial pile displacements under service limit state conditions. However, using the axial shear transfer data derived using methods as presented in 8.4 (in particular, equating t_{max} with $f(z)$ in clay soils) will not produce the representative axial capacity under ultimate loading conditions conditions if the pile is axially flexible and the soil exhibits strain softening.

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In some circumstances (e.g. for soils that exhibit strain-softening behaviour or where long piles can be axially flexible), the axial capacity of the pile should be derived explicitly accounting for the post-peak degradation of the unit skin friction at large strain.

A.8.4.2 End bearing resistance–displacement, *Q–z* curve

No additional guidance is offered.

A.8.5 Soil reaction for piles under lateral actions

A.8.5.1 General

Lateral soil resistance–displacement *p–y* curves should be constructed using stress–strain data from laboratory soil samples. The ordinate for these curves is soil resistance, *p*, and the abscissa is the pile wall displacement, *y*. By iterative procedures, a compatible set of lateral resistance-displacement values for the pile-soil system can be developed.

More detailed study of the construction of *p–y* curves can be found in the following sources:

a) Reference [184] and [185] for monotonic curves in clays;

b) References [368] for cyclic curves in clays;

c) Reference [358] for fatigue curves for clays and sands;

d) Reference [259] for monotonic and cyclic curves in sand;

e) Reference [138] for layered soils.

A.8.5.2 Lateral soil reaction for clay

A.8.5.2.1 General

Reference [184] describes the numerical analyses and the extensive database of 537 DSS tests that were used to develop the p-y curve framework. It also details the hindcast of eleven pile load tests which demonstrated the wide range of applicability of the method.

A.8.5.2.2 *p–y* curves for monotonic actions

A.8.5.2.2.1 General

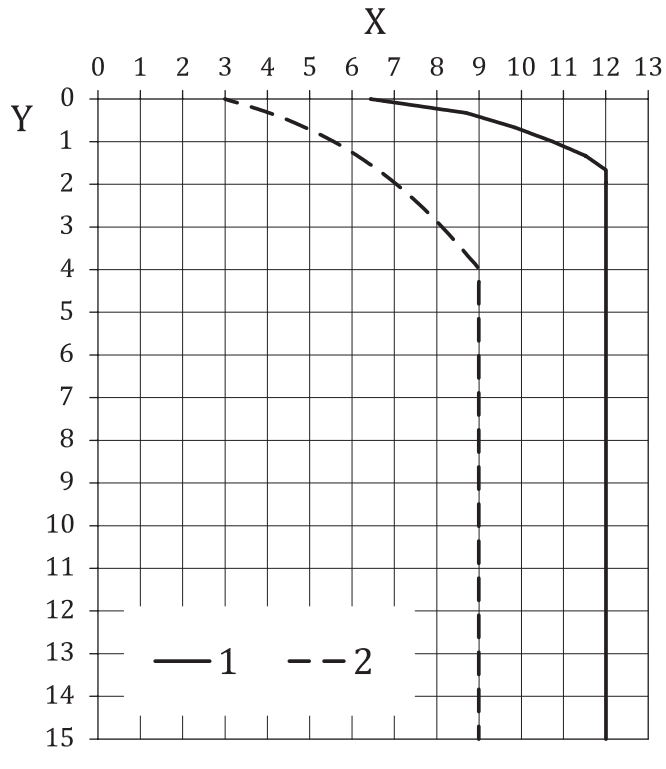
No additional guidance is offered.

A.8.5.2.2.2 Ultimate soil resistance for isotropic conditions

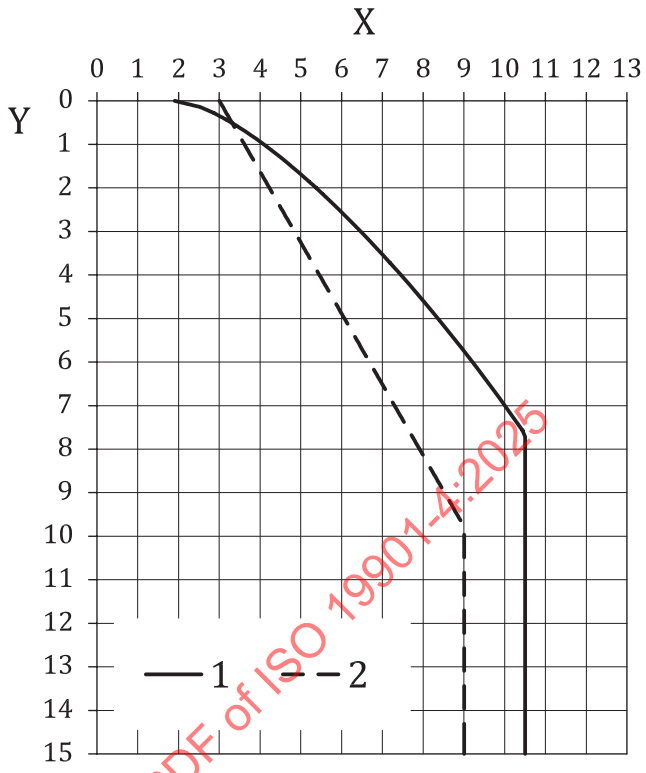
The recommended values of p_u will generally be higher than those of API RP 2GEO 1st edition as shown in Figure A.13. Extensive comparisons between various recommendations can be found in Reference [230].

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Example 1



Example 1

Example	s_{uo} (kPa)	s_{u1} (kPa/m)	D (m)	γ (kN/m³)	α (-)	$\frac{s_{uTE}}{s_{uDSS}}$	Gap	J (ISO 19901-4:2016. Only)
1	2	1,2	0,914	7	1	0,9	No	0,5
2	150	0	2,134	8	0,5	0,7	Yes	0,5

Key

X lateral bearing capacity factor, N_p (-)
Y normalised depth, z/D (-)
1 this document
2 ISO 19901-4:2016

Figure A.13 — Comparison of total lateral bearing capacity factor N_p according to this document and ISO 19901-4:2016 for example 1 (left) and example 2 (right)

A.8.5.2.2.3 Anisotropy correction of ultimate soil resistance for gapping conditions

Two types of anisotropy are considered in the p-y curve.

— Difference in strength between the triaxial extension and DSS failure modes.

a) Detailed discussion on anisotropic clay properties can be found in References [230] and [212] for GoM clays, and Reference [18] for Drammen clay.

b) Anisotropy considerations are usually not warranted for very soft clays close to the seafloor because:

c) a laterally loaded pile in such clays will typically behave in a non-gapping condition;

d) the DSS shear strength is typically a reasonable estimate of the average of the triaxial compression shear strength and the triaxial extension shear strength (as is the case for GoM clays).

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e) In the flow-around mechanism (i.e. when $N_p = N_{pd}$), the value of p_u is solely based on the DSS strength and needs no correction for anisotropy between the triaxial and DSS failure modes.

— Difference in DSS strength on a vertical plane and a horizontal plane in the flow-around mechanism.

f) In principle, the DSS tests should be performed on vertically trimmed samples to measure the strength on the *in- situ* vertical plane and better simulate the flow around conditions. However, industry practice is to perform DSS tests on horizontally trimmed samples and measure the strength on the in situ horizontal plane.

g) Data suggest that the ratio of the strength on the in situ horizontal plane over the strength on the in situ vertical plane is close to unity for clays with I_p greater than 30 % and therefore no correction is needed when measuring s_u on a horizontally trimmed sample. The influence of this inherent anisotropy should be evaluated for I_p < 30 %. More guidance can be found in Reference [230].

For highly non-linear profiles (e.g. profiles with a crust near the seafloor or a large step-increase in shear strength), guidance on how to calculate the bearing capacity factor N_p can be found in References [152] and [151].

A.8.5.2.2.4 *P-y* curve relationships

Guidance on how to scale laboratory DSS stress-strain curves to obtain *p-y* curves can be found in Reference [230]. Normalized *p-y* curves, (p/p_u) vs (y/D), are obtained from normalized DSS curves, (τ/s_u) vs γ.

The default normalized curves of Table 2 for I_p greater than 30 % were obtained by substituting the following values in Method 2 of Figure A.14 to obtain a set of three normalized DSS curves:

— For OCR ≤ 2: γ_f = 0,15; a = 2,38;

— For OCR =4: γ_f = 0,15; a = 1,5;

— For OCR = 10: γ_f = 0,15; a = 1,0.

Each of these normalized DSS curves was then scaled twice as per method 2 of Figure A.12 and as per the full procedure as described in Reference [367] to obtain two *p-y* curves. Each of the default normalized curves of Table 2 is a best fit through these two obtained *p-y* curves.

The default normalized curves of Table 2 for I_p less than 30 % were obtained, based on experience with Drammen clay, by multiplying the (y/D) abscissa on the normalized curves for I_p greater than 30 % for the same OCR as follows:

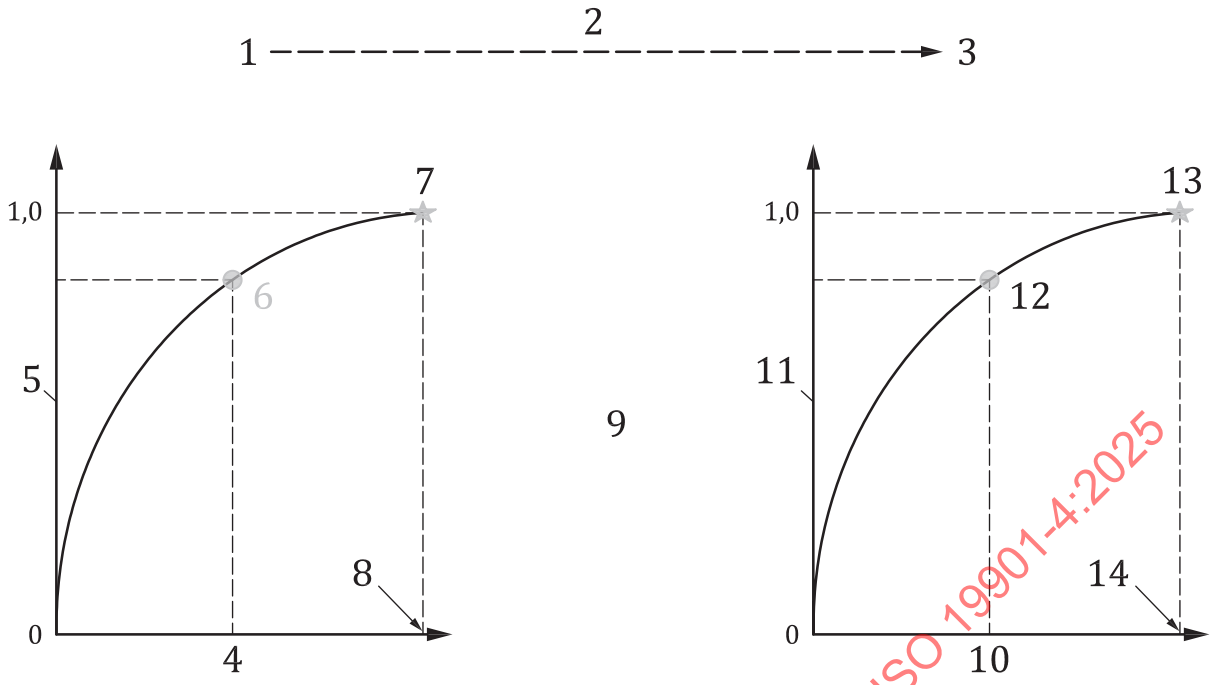
— For OCR ≤ 2: (y/D) abscissa multiplied by 0,33;

— For OCR =4: (y/D) abscissa multiplied by 0,5;

— For OCR = 10: (y/D) abscissa multiplied by 0,66.

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Key

1	normalized DSS curve, up to peak shear stress	8	γ_f
2	scaling	9	see footnote A below
3	normalized *p-y* curve	10	normalized pile displacement, y/D
4	shear strain, γ	11	normalized soil resistance, p/p_u
5	normalized shear stress, τ/s_u	12	$(y/D\ ,\ p/p_u)$
6	$(\gamma,\ \tau/s_u)$	13	failure
7	failure	14	$(y/D)_f$

Footnote A: two methods to obtain *p-y* curves from DSS curves:

Method 1:

$p/p_u = \tau/s_u$

$y/D = (1{,}35 + 0{,}25\ a)\ \gamma$

Method 2:

1. Determine strain at failure, γ_f
2. Curve fit normalized DSS curve with Eq. 1 below and obtain parameter "a", which typically ranges from 1,0 to 3,5.

$$\frac{\tau}{s_u} = \frac{\tanh\left[a \cdot \left(\frac{\gamma}{\gamma_f}\right)^{0,5}\right]}{\tanh(a)}$$

3. Obtain normalized p-y curve as

$$\frac{p}{p_u} = \frac{\tanh\left[A \cdot \left(\frac{y/D}{y/D_f}\right)^{0,5}\right]}{\tanh(A)} h$$

With $A = 1{,}33 + 0{,}45\ a$ and $(y/D)_f = \gamma_f\ (2{,}5 - 1{,}2\ \ln(a)\ )$

Figure A.14 — Methods to obtain *p-y* curves from DSS curves

A.8.5.2.3 *P-y* curves for cyclic actions

The full method to develop cyclic *p-y* curves for GoM and North Sea conditions can be found in References [369] and [368]. Validation of the method and comparison with cyclic pile load test results can be found in Reference [368].

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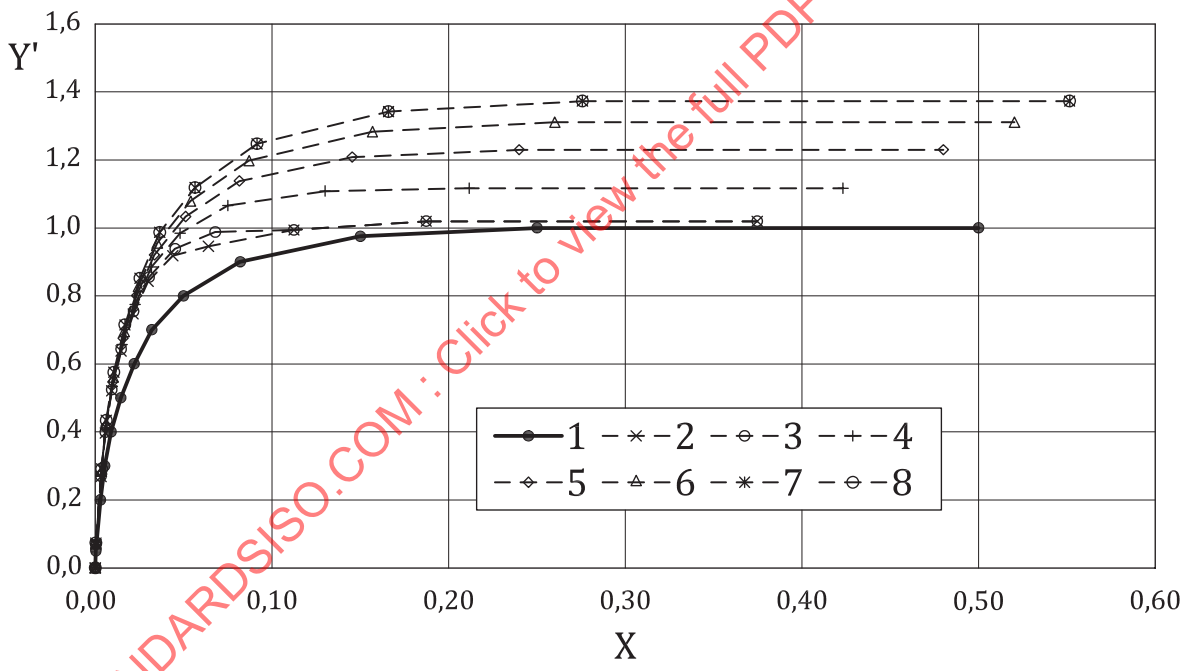
The p-y curve method includes an iterative process and requires knowledge of the design action and the detailed pile characteristics. The method in the main text is a simplified approach and is intended to moderately overestimate cyclic effects.

Contrary to previous recommendations in ISO 19901-4:2016, the cyclic curves are stiffer and have higher ultimate resistance than the monotonic curves. This arises in part because the previous recommendations were developed from a single series of tests performed with 200 cycles to 400 cycles at each lateral loading increment and were intended to provide a lower bound of soil resistance.

In contrast, the new recommendations provide best-estimate soil resistance for loadings acting on jacket structures. The number of equivalent cycles varies with depth and with lateral displacement, but is less than 25 for such structures. For these low number of cycles, the soil cyclic strength is greater than the reference monotonic strength, and the *p*-modifier is greater than 1,0.

The normalized cyclic curves vary along the depth of the pile. An example calculation as per 8.5.2.2 is presented in Figure A.15 for an illustrative clay soil profile with I_p > 30 %, OCR < 2, and for GoM cyclic loading conditions.

s_{uo} (kPa)	s_{u1} (kPa/m)	D (m)	γ (kN/m^3)	α (-)	$\frac{s_{uTE}}{s_{uDSS}}$	Gap	Estimated pile rotation depth (m)
2	1,2	2,134	7	1	0,9	No	32



Key

X	normalized lateral displacement, (y/D)	4	cyclic curve, Z/Z_{rot} = 0,4
Y'	normalized soil resistance, (p/p_u)	5	cyclic curve, Z/Z_{rot} = 0,6
1	monotonic curve for I_p > 30 % and OCR < 2	6	cyclic curve, Z/Z_{rot} = 0,8
2	cyclic curve, Z/Z_{rot} = 0	7	cyclic curve, Z/Z_{rot} = 1,0
3	cyclic curve, Z/Z_{rot} = 0,2	8	cyclic curve, Z/Z_{rot} = 1,1

Figure A.15 — Cyclic normalized curves for illustrative clay soil profile with I_p > 30 %, OCR < 2, and for GoM cyclic loading conditions

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A.8.5.2.4 *P-y* curves for fatigue actions

Development and validation of the p-y curves for fatigue actions can be found in Reference [363] where hindcast of conductor/wellhead/BOP/LMRP motions are compared with those predicted using the recommended *p-y* curves.

A.8.5.2.5 *P-y* curves for earthquake actions

Reference [214] details the results of centrifuge tests which showed that, for a jacket in soft clay, the structural natural period and bending moment profile along the piles were accurately predicted with:

a) the *p-y* curves derived as per Reference [180];

NOTE The recommendations of 8.5.2.1 are an update from Reference [180] and give similar results for OCR < 2 and I_p > 30 %, non-gapping clays with a skin friction factor, α, equal to 1,0.

b) hysteretic unload-reload behaviour;

c) lateral and axial radiation damping;

d) depth-varying ground motion input.

A.8.5.2.6 Comparison with previous recommendations

The monotonic, cyclic and fatigue curves in Clause A.8.5.2 are compared with the monotonic and cyclic curves recommended by ISO 19901-4:2016 in Figure A.16 for an illustrative soil profile and GoM conditions. The input parameters for the comparison are given in Table A.4.

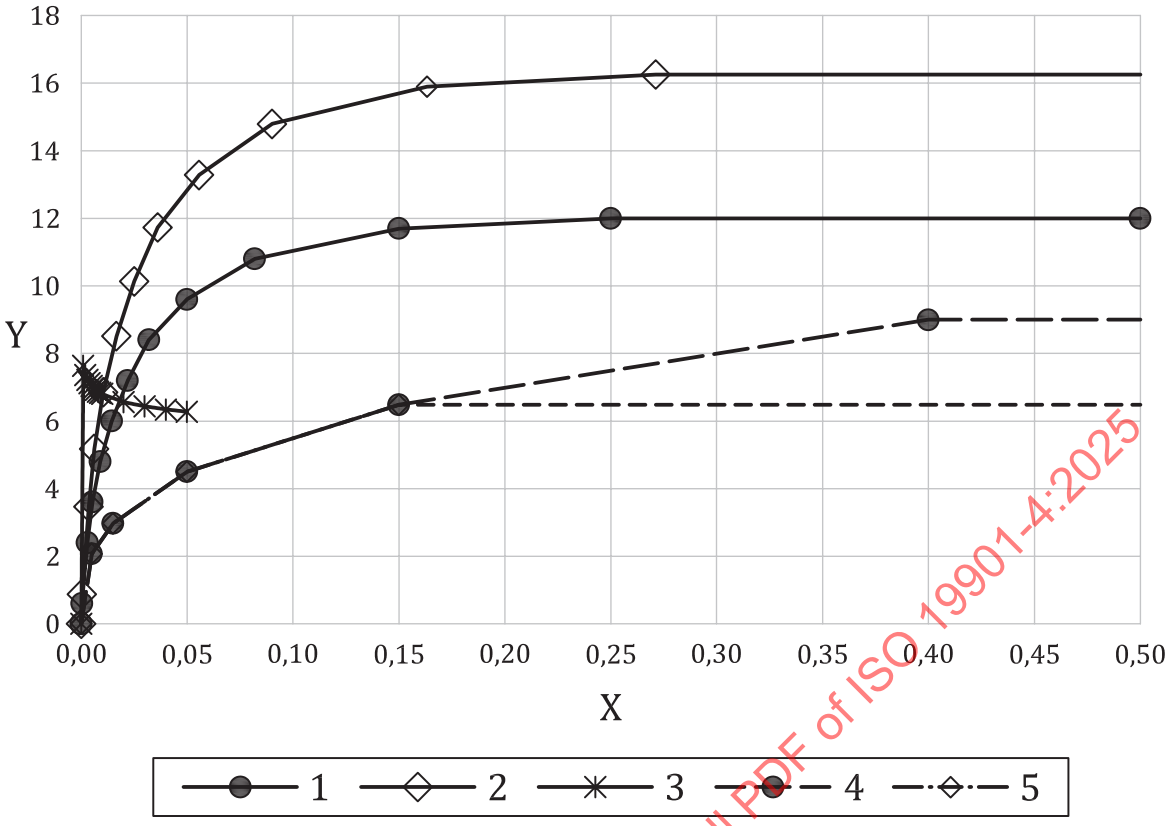
Table A.4 — Input parameters for comparison of *p-y* curves

s_{uo} (kPa)	s_{u1} (kPa/m)	D (m)	γ (kN/m^3)	α (-)	$\frac{s_{uTE}}{s_{uDSS}}$	Gap	Estimated rotation depth (m)	ε_c for API RP 2GEO 1st edition method for soft clays
2	1,2	2,134	7	1	0,9	No	32	0,02

The vertical axis on Figure A.16 is labelled 'p/s_uD' and is unitless with the definition of 'p' as a force per unit length. The unit of 'p' is now consistent with the units for the *p-y* curves in sands. In ISO 19901-4:2016, 'p' had a unit of pressure and 'P' had a unit of force per unit length. Therefore, the values calculated with ISO 19901-4:2016 as shown in Figure A.16 are actually 'P/s_uD' when using the definitions in that document.

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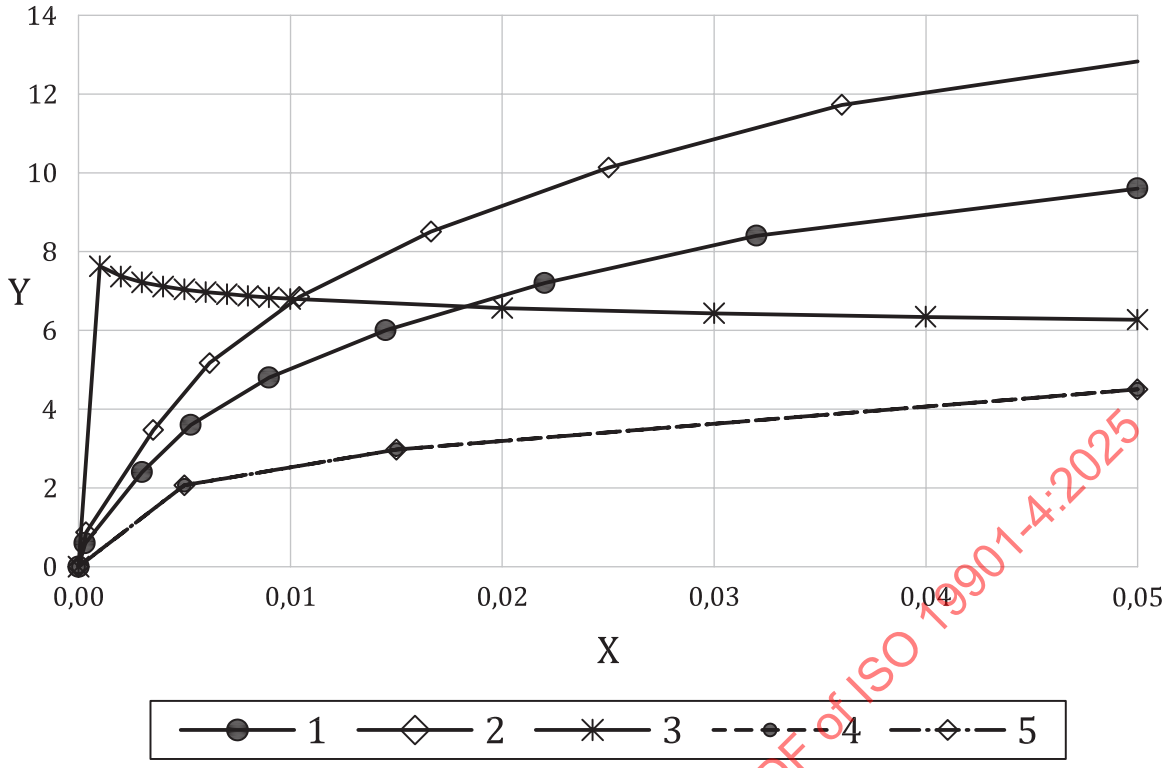
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a) Comparison of *p-y* curve recommendations, full scale

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b) Comparison of *p-y* curve recommendations, low displacements

Key

X	normalized lateral displacement (y/D)	3	fatigue curve, this document
Y	normalized soil resistance (p/s_uD)	4	monotonic curve – ISO 19901-4:2016
1	monotonic curve, clause 8.5.2.2, I_p > 30 % and OCR < 2	5	cyclic curve – ISO 19901-4:2016
2	cyclic curve, clause 8.5.2.3., GoM condition		

Figure A.16 — Comparison of *p-y* curve recommendations between ISO 19901-4:2016 [174] and this document at a depth *z* = 30 m for illustrative clay soil profile for GoM conditions: a) full scale and b) low *y/D* displacements

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A.8.5.3 Lateral capacity for sand

Scour (i.e. seabed sediment erosion due to wave and current action) can reduce lateral soil support around offshore piles, leading to an increase in pile maximum bending stress. Scour is generally not a concern for clay soils, but should be considered for sands.

In the absence of project specific data, for an isolated pile a local scour depth equal to 1,5D and an overburden reduction depth equal to 6D may be adopted, D being the pile outside diameter; see Figure A.17.

Reduction in lateral soil support is due to two effects:

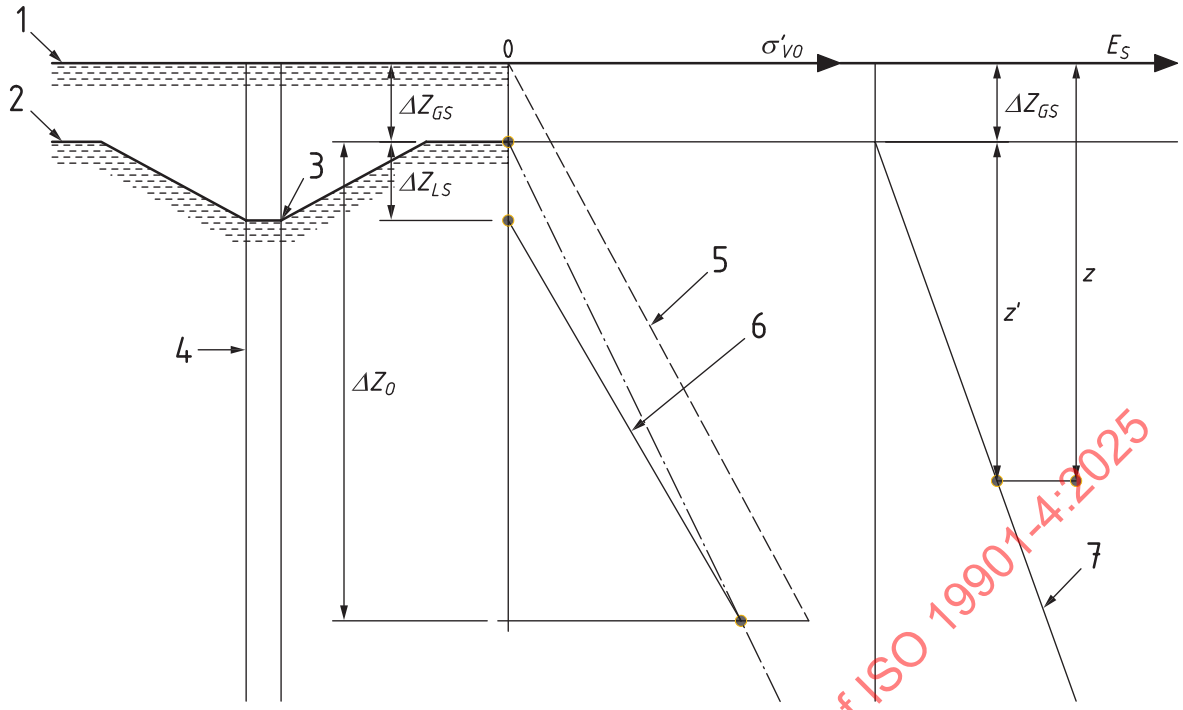
a) a lower ultimate lateral pressure caused by decreased vertical effective stress, $\sigma'_{vo}(z)$;

b) a decreased initial modulus of subgrade reaction, E_S.

There is no generally accepted method to allow for scour in the p–y curves for offshore piles. Figure A.15 suggests one of the methods for evaluating $\sigma'_{vo}(z)$ and E_S as a function of scour depths. In this method, general scour reduces the $\sigma'_{vo}(z)$ profile uniformly with depth, whereas local scour reduces $\sigma'_{vo}(z)$ linearly with depth to a certain depth below the base of the scour pit. Subgrade modulus reaction values, E_S, can be computed assuming the general scour condition only.

Other methods, based upon local practice, model testing [180] and/or experience, can be used instead.

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Key

1	original seafloor level	Δz_{GS}	global scour depth
2	level after general scour	Δz_{LS}	local scour depth (1,5*D* typical)
3	level of local scour	Δz_0	overburden reduction depth (6*D* typical)
4	pile	σ'_{vo}	vertical effective stress
5	no scour case	E_S	initial modulus of subgrade reaction
6	local scour case	z	depth below original seafloor
7	$E_s = k\,z'$ (see Table 3 for k)	z'	final depth below seafloor, after general scour

Figure A.17 — *p-y* lateral support: scour model

A.8.5.4 Lateral soil resistance-displacement *p-y* curves for sand

No additional guidance is offered.

A.8.5.5 *p–y* curves for fatigue actions

No additional guidance is offered.

A.8.5.6 Refined assessment of lateral pile response

For intermediate foundations where the pile or caisson *L/D* ratio is low (i.e. less than about 10), or when the design is sensitive to lateral actions, analysis methods which involve the definition of case-specific soil reaction curves defined through, for example, finite element analyses are available.

References [1] and [62] describe the 'PISA' 1D modelling procedure which recognizes that further soil reactions develop in addition to the lateral soil reactions considered in *p-y* calculations. As illustrated in Figure A.18, four separate soil reaction components are considered at the soil-pile interface: (i) distributed lateral actions, (ii) vertical shear tractions combining to form a moment, (iii) a horizontal force at the pile base, and (iv) a moment at the pile base.

Each of the soil reaction components is related in the model to the local lateral displacement or rotation (i.e. adopting a 'Winkler' approach) by a calibrated parametric function referred to as a 'soil reaction curve'. In the computational implementation (the 1D model illustrated in Figure A.16), the pile is represented as an

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embedded beam, which is best represented by Timoshenko beam theory. A distributed lateral action p and a distributed moment m are assumed to act along its length. The distributed moment represents the moments associated with the vertical shear tractions induced at the soil-pile interface. Additionally, a horizontal force H_B and a moment M_B act on the pile base. Figure A.19 sets out two alternative processes that can be followed to define the sets of soil reaction curves.

a) Rule-based method: This approach employs pre-defined curves that are either in the wider literature or can be derived through analytical methods. Examples of such curves include existing published formulations for the p-y method (see Reference [230]) or new bespoke sets of all four soil reaction curves for glacial clay till (see Reference [63]) and marine dense sand (see Reference [59]) that are linked to soil parameters acquired through appropriate soil investigations. Application to layered soil profiles is addressed in References [62] and [58].

b) Numerical-based method: This approach adopts finite element calculations to generate and calibrate site-specific or regional soil type reaction curves that can be used in the 1D model calculations. This requires the following steps.

   1) A characterization of the site ground model and the properties of the soils present that is sufficiently thorough to inform accurate non-linear modelling of their non-linear stiffness and failure behaviour. This will usually require detailed knowledge of undrained strength in clays, relative density and CPT cone resistance in sands and reliable information on soil stiffness (see Reference [365]).

   2) Specification of the soil investigation and laboratory testing program to provide the data required for step 1).

   3) Calibration of the soil constitutive model used for the finite element calibration analysis, based on the information obtained in step 2) [320].

   4) A specification of the design parameter space for the finite element analyses, within which the soil reaction curves will be calibrated (e.g. pile geometry, loading conditions), and over which the calibrated 1D model is valid.

   5) A choice of the functional form of the non-linear curves used as the basis function for the soil reaction curves, calibrated via data abstracted from the finite element analyses.

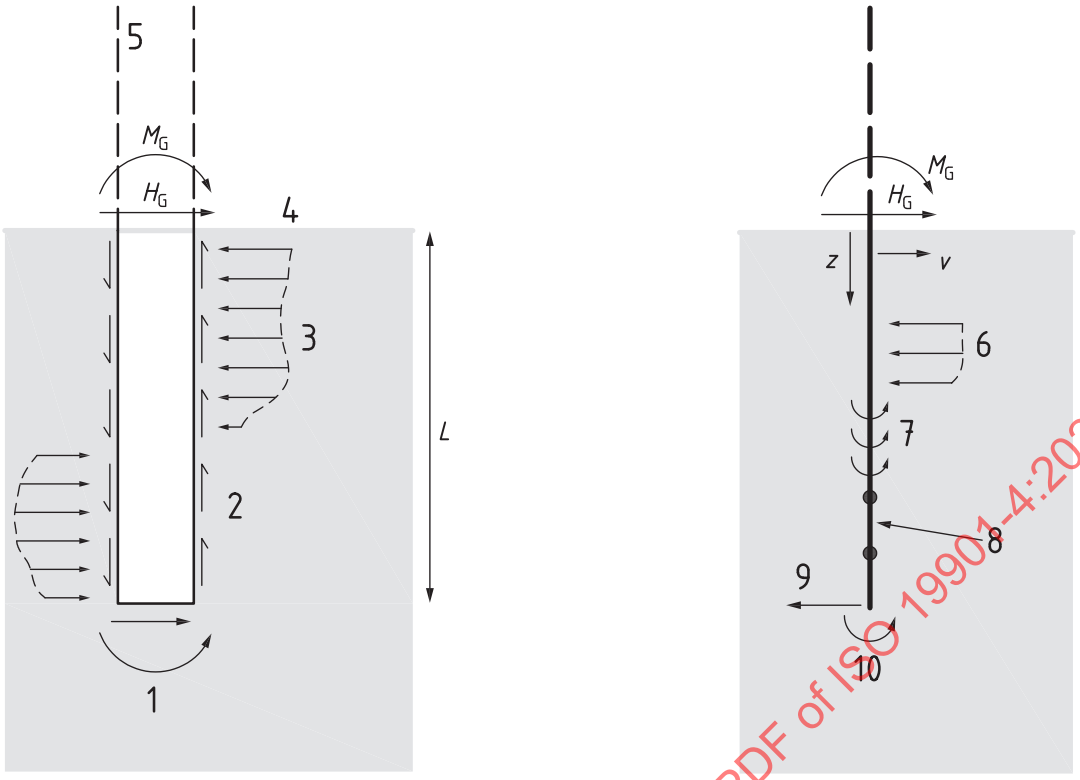
   6) A procedure for abstracting the soil reaction curve data from the finite element analyses, and providing an optimization of fit of the soil reaction curve basis function against the abstracted data from the finite element analyses.

   7) Development of a scaling procedure that allows the soil properties from step 1) to be input to steps 5) and 6) so that the derived soil reaction curves can be more widely applied in the simplified 1D design model across the proposed design space.

Option a) may be adopted for design where site investigation data are limited, or at an early stage of the design process. Option b) may be incorporated as part of the detailed design procedure. Implementation of option b) at different sites will increase, over time, the spread of cases for which calibrated soil reaction curves are available to apply option a). Further details of the method can be found in References [1], [62], [63], [59] and [58].

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a) soil reaction components acting on a monopile **b) soil reaction in 1D finite element implementation**

Key

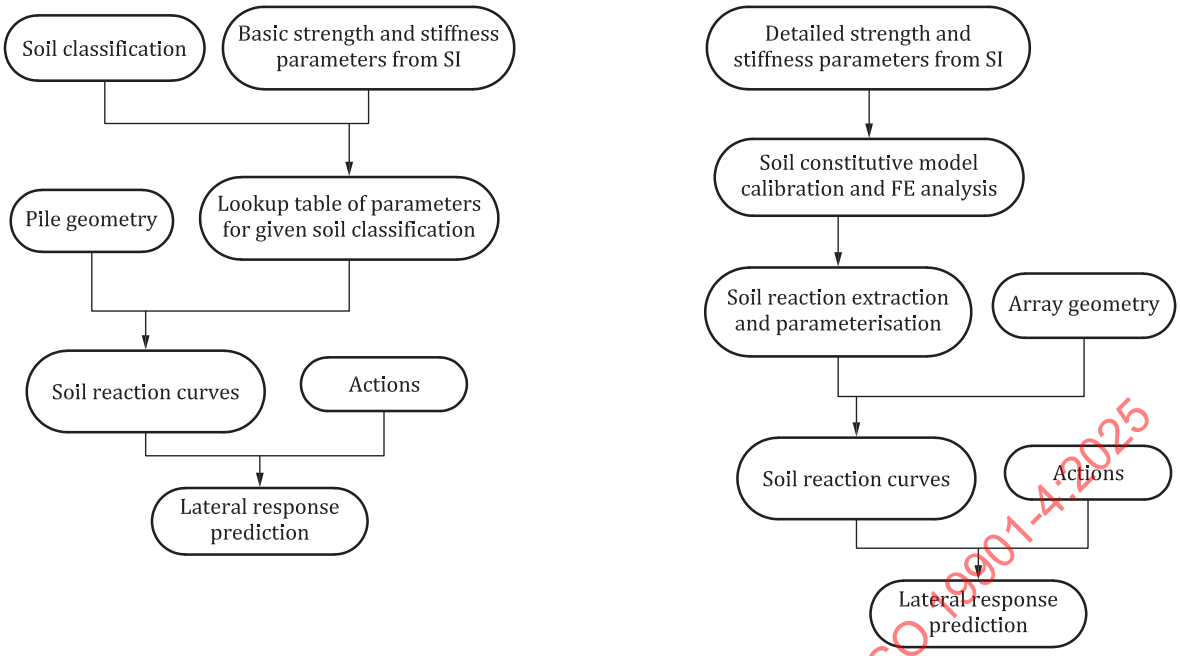
1	horizontal force and moment applied at the pile base	6	distributed lateral load $p(z,v)$
2	vertical shear force tractions at soil-pile interface	7	distributed moment $m(z,\psi)$
3	distributed lateral load	8	timoshenko beam elements
4	ground level	9	base horizontal force $H_B(V_B)$
5	tower	10	base moment $M_B(\psi_B)$

NOTE The reactions are depicted in sub-figure a) as acting in the expected direction. In sub-figure b) the reactions are shown in directions that are consistent with the coordinate directions shown.

Figure A.18 — Soil reaction components acting on a monopile and 1D finite element model

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a) Rule-based method **b) Numerical-based method**

Figure A.19 — Application modes for the design method

A.8.6 Pile group behaviour

A.8.6.1 General

Routine numerical analysis of pile groups can be divided into two main categories.

The first category, which is computationally the simplest, uses algebraic expressions to define the elastic single pile resistance to general (axial, lateral and torsional) actions [268]. The group resistance is determined by modifying the single pile expressions to account for elastic pile-soil-pile interaction.

The second category, which is normally performed for offshore pile groups, is more rigorous. Methods are usually hybrid, employing a mixture of discrete *p*–*y* curves (Winkler approach) and continuum soil behaviour, first described in Reference [128] for lateral analysis. Since then, numerous programs have been developed worldwide for general types of action. Typically, the nonlinear single pile resistances to general actions are computed using axial *t*–*z* and lateral *p*–*y* curves and combined with elastic interaction expressions similar to the first category. The resulting formula are solved for various pile head fixity conditions and/or pile cap restraint to determine the nonlinear group resistance and individual pile forces and moments, plus the so-called '*z*- and *y*-modifiers'.

References [268] and [255] provide more detailed discussions.

A.8.6.2 Axial behaviour

In general, group effects depend considerably on pile group geometry and penetrations and thickness of any bearing stratum underneath the pile tips [268], [255].

A.8.6.3 Lateral behaviour

Experience confirms that the available tools for analysis of pile groups subjected to lateral actions provide approximate answers that sometimes deviate significantly from observed behaviour, particularly with regard to displacement calculations. Also, limitations in soil investigation procedures and in the ability to predict soil-pile interaction behaviour for a single pile produce uncertainty regarding proper soil input to

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group analyses. Therefore, multiple analyses should be performed for pile groups using two or more methods of analysis and upper estimates and lower estimates values of soil properties in the analyses. By performing such analyses, the designer will obtain an appreciation for the uncertainty involved in his predictions of foundation performance and can make more informed decisions regarding the structural design of the foundation and structure elements.

A.8.7 Pile installation assessment

A.8.7.1 General

Drivability studies are carried out in accordance with the principles given in 8.7.2 and A.8.7.2 in order to define the type of hammer necessary to reach the target design pile penetration. The design penetration of driven piles should not be determined upon any correlation of pile capacity with the number of blows required to drive the pile a certain distance into the seabed.

Vibratory hammers can be considered for installing well conductors, or piles which are predominantly subjected to horizontal actions, such as reaction piles for start-up of pipelines or anchor piles. They can also be used where extraction and repositioning can be required. Vibratory hammers can further be considered as complementary tools to impact hammers, i.e. for initial driving [187].

In order to minimize delays in installation, a pile acceptance procedure should be established. The procedure should outline the measures to be taken on location for adjusting planned pile driving scenarios, in case of, for example, premature pile driving refusal or a significantly lower blow count than anticipated at design target pile penetration.

A.8.7.2 Drivability studies

Drivability studies are required to cover a wide range of soil types and driving conditions and should take into account local experience. The principles of drivability are presented in References [309] and [330], with friction fatigue effect discussed in Reference [157] and damping parameters presented in Reference [293]. Soil resistance to drive for a variety of different soil types and driving conditions are defined in Reference [6]. For Gulf of Mexico clays, guidance is given in Reference [364] and more specifically in Reference [110] for tension leg platform piles and in Reference [115] for deep water cases. For North Sea cases, guidance is given in Reference [275] and in References [6] and [119] for hard clays. For West Africa soils, guidance is given in Reference [87] and in Reference [270] for soft carbonate rocks.

A.8.7.3 Obtaining required pile penetration

No additional guidance is offered.

A.8.7.4 Driven pile refusal

The following are two examples of driven refusal criteria.

a) In soft soils, pile driving refusal for a properly operating hammer is defined as the point where pile driving resistance exceeds either 1 000 blows/m (330 blows/ft) for a consecutive 1,5 m (5 ft) of penetration, or 800 blows for 300 mm (1 ft) of penetration. This definition applies when the weight of the pile does not exceed four times the weight of the hammer ram. If the pile weight exceeds this, the above blow counts are increased proportionally, but in no case should they exceed 800 blows for 150 mm (6 in) of penetration.

b) In hard clays and dense sands, pile driving refusal can be defined as the point where driving resistance exceeds one of the following criteria:

— in continuous driving, a minimum of 125 blows/250 mm (165 blows/ft) over 6 consecutive intervals of 250 mm, or a minimum of 200 blows/250 mm over 2 consecutive intervals of 250 mm;

— in the last interval of 250 mm at the end of driving, 325 blows/250 mm (400 blows/ft);

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— at restart of driving after a stoppage for 1 h or longer, 325 blows/250 mm over 2 consecutive intervals of 250 mm.

In soils where hard driving conditions are anticipated, such as in the presence of boulders or of strong cemented layers, the definition of pile refusal criteria cannot be based solely on a blow count value, and the potentially high local driving stresses induced in the pile should also be taken into account. The stress level in the pile steel can be calculated from wave formula analyses, and can be estimated from the stress measurements from pile instrumentation. An example of refusal criteria for pile driving in strongly cemented carbonate soils is given in References [316] and [349].

The potential consequences of hard driving conditions in strong cemented layers (i.e. damage of the pile, hammer or structure) are highly dependent on the hammer type and size, on the pile wall thickness (D/WT ratio, presence of a driving shoe), and on possible defects and irregularities in the pile shape, as well as on the soil conditions (in particular, strength and thickness of the rock layer, and soil type below the rock formation). Moreover, the reflected stress level (ratio of the maximum reflected stress to the initial peak stress), as measured from pile instrumentation at the pile head, only gives an estimate of the average stress in the pile wall; more severe stresses can be experienced locally at the pile tip during driving. Therefore, the definition of driven pile refusal criteria in cemented soils should preferably be based on local piling experience at the site. Correlation charts, similar to the one proposed in Reference [270], can be developed as an aid in deciding whether pile driving through a cemented layer can be attempted, or if drilling of the rock below the pile tip is necessary.

A.8.7.5 Pile refusal remedial measures

No additional guidance is offered.

A.8.7.6 Selection of pile hammer and stresses during driving

The designer should be aware that pile buckling and pile refusal incidents in very dense sands have been associated with the use of external chamfers at the pile tip. Although factors other than the shape of the pile tip contribute to buckling, the use of an external chamfer can increase the potential for buckling and/or refusal [57][168].

A.8.7.7 Use of hydraulic hammers

No additional guidance is offered.

A.8.7.8 Drilled and grouted piles

No additional guidance is offered.

A.8.7.9 Grouting pile-to-sleeve connections

The equipment should have sufficient capacity to achieve the grout filling in a single continuous operation. Grouting should not commence unless there are sufficient materials available, including a contingency, to complete the task. Grout slurry stored in a holding tank should be continuously stirred and should not be held for more than 30 min prior to pumping. In case of rapid hardening mixes, the storage duration in a holding tank should be reduced to less than 30 min. Further details concerning quality control as well as requirements for conducting grout trials that address cement grout for connections and repairs can be found in ISO 19902.

Prior to grouting and after activating any sealing devices, dyed water should be flushed through the complete grouting system to both remove any deleterious matter and to prove its functionality. A pressure test can be appropriate for closed systems. The annulus should then be carefully filled by maintaining a continuous grout flow through the lowest practical point.

Grout returns to allow surface sampling are preferable. If these are provided, grout samples for strength conformance testing can be taken from the returns in addition to the slurry specific gravity measurements. If

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surface returns are not provided, visual inspection to confirm that grout has completely filled the annulus should be performed immediately after cessation of grout pumping and again after initial grout set, typically 12 h.

A.8.7.10 Pile installation data

No additional guidance is offered.

A.8.7.11 Installation of conductors and shallow well drilling

Additional guidance about the installation of conductors by driving is provided in A.10.5.6.

A.9 Guidance on assessment of pile capacity for existing structures

A.9.1 General

In accordance with ISO 19901-9:2019, Clause 9 is limited to the capacity reassessment for driven piles and conductors making the foundation system of fixed steel offshore structures.

A.9.2 Geotechnical and foundation data

No additional guidance is offered.

A.9.3 Evaluation

No additional guidance is offered.

A.9.4 Assessment

A.9.4.1 General

No additional guidance is offered.

A.9.4.2 Pushover response of pile foundation systems

The following guidance is offered for how the pile foundation and conductor system should be treated in pushover analyses.

a) Effects of cyclic actions: The cyclic stress-strain behaviour of the soil can be assessed from a series of laboratory tests or centrifuge tests, which can then be used in an analysis together with the cyclic load history to evaluate the performance of the whole foundation system. Cyclic pile loading tests can also be used as reference to understand the likely response during a pushover analysis. Additional guidance about the cyclic behaviour of piles is provided in A.8.3.2.

b) Conductors: The platform and conductor guide framing that constrains and engages the conductors should be modelled so that the lateral displacements of the conductors at the seafloor are consistent with the behaviour of the overall structure under a given loading condition, and consistent with the load levels at which the conductor guide framing is expected to fail.

Other risk implications than for a pile foundation should be considered, if the conductors fail before the platform system. These are not considered in this document; specific advice from well specialists should be sought in these cases.

The limited available information to date suggests that the performance of platform foundation systems in hurricanes is consistent with predictions based on design capacities [139] without the need to increase the shear strength of the soil to account for unexpected survivals of pile foundation systems. When analysing a pile foundation system survival in an extreme loading event, the following factors should be addressed.

— If the axial capacity of the pile is mostly due to sands acting in skin friction or end bearing, the undrained shear strength of clay layers alone will have very little impact on the axial capacity.

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— If sands are present, CPT-based methods will provide more reliable estimates of pile capacity (see 8.1.4).

— If the pile system is failing in shear, the capacity of the system is much more sensitive to the bending moment capacity of the piles and conductors than to the shear strength of the soil. A relatively small increase in bending moment capacity, such as with an average versus a nominal steel yield stress, can have a greater effect on the capacity of the pile system.

— The shear strength of the soil is being used as a convenient surrogate for lateral and axial soil resistance, since the soil shear strength can be changed easily as input to a pushover analysis. However, the relationship between lateral or axial soil resistance and shear strength is not directly proportional. Increasing the undrained shear strength of clay layers causes them to be treated as more heavily over-consolidated in the design recipe for axial side shear. In this case, the greater the undrained shear strength, the less sensitive the axial capacity will be to an increase in the undrained shear strength.

When a CPT-based method is used for predicting the pile capacity in a pushover analysis, consistent axial response of the pile foundation system as represented by shear and end bearing transfer *t-z* and *Q-z* curves should be defined, i.e.:

— the shape of the *t-z* and *Q-z* curves can be defined in accordance with 8.4.1 and 8.4.2, but with the maximum soil-pile unit skin friction, t_{max}, at any depth along the pile length, and the representative end bearing resistance, Q_p, determined with the CPT-based method;

— an appropriately brittle *t-z* curve should be adopted.

A.9.5 Time-dependent effects on pile foundations

Ageing over a period of about one year leads to significant gains in the shaft friction of driven piles in sands in first-time loading. Smaller gains have been observed for driven piles that are re-tested and, by inference, working piles that experience high levels of cycling. Until such time as the mechanisms of ageing are better understood, and if field measurements confirming ageing effects on piles or conductors in similar conditions do not exist, adopting the lower bound ageing characteristics presented in Reference [136] can be considered.

Sensitivity analyses should be performed where increases in pile shaft capacity due to ageing can result in structural issues.

A.10 Guidance on geotechnical design input to subsea structures, risers and flowlines

A.10.1 General

No additional guidance is offered.

A.10.2 Geotechnical investigation

No additional guidance is offered.

A.10.3 Foundations for subsea production structures

No additional guidance is offered.

A.10.4 Steel catenary risers

A.10.4.1 General

Riser interaction with the seabed involves processes such as plastic penetration during initial touchdown, softening during cycles of upward and downward motion and potential suction-induced tensile resistance prior to breakaway, water entrainment into seabed, and soil erosion. In most cases, design is undertaken using simplified models where the riser–soil interaction is idealized by a series of springs with zero tension capacity distributed along the riser throughout the touchdown zone. The choice of spring stiffness should be based on the amplitude of vertical displacement and other effects such as the cyclic motion of the riser. While

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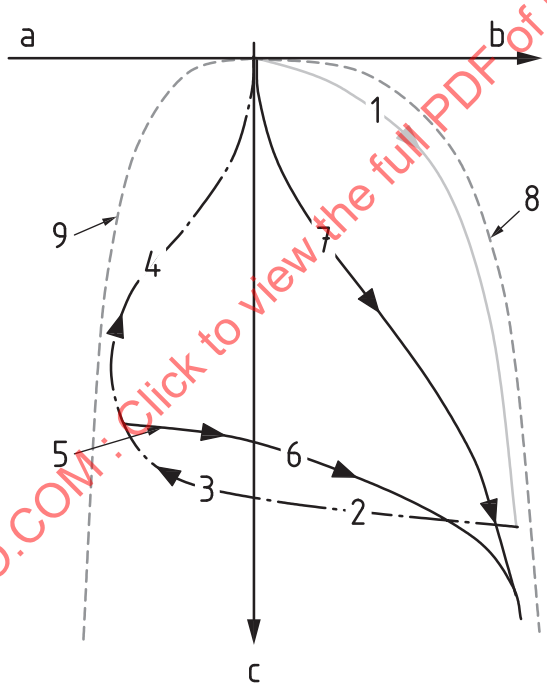
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the soil response will also be affected by out-of-plane motion of the riser, the discussion in this subclause is restricted to vertical stiffness of the seabed.

The conceptual description of the seabed resistance is shown in Figure A.20 for a few of the processes involving soil–riser interaction. Following initial riser penetration into the seabed, unloading occurs as the pipe is uplifted. The soil response in the early stages of uplift is stiffer than that under conditions of virgin penetration as shown in the 'unloading' curve in Figure A.20. With continued uplift, the net resistance force goes into tension ('pipe-soil suction' in Figure A.20) until maximum uplift resistance of the soil is reached and the pipe begins to detach from the soil. Uplift resistance decreases until the pipe completely detaches from the soil. Upon re-penetration, the pipe comes back into contact with the soil, with the re-loading stiffness typically being less than the unloading stiffness. Upon completion of a full action cycle, the action path does not return to the initial point of departure from the backbone curve; rather the pipe penetrates a small additional depth into the soil. The effects from cyclic loading will reduce the stiffness upon subsequent cycles although long term consolidation can result in an increase in soil stiffness.

The information provided in this subclause for motions in the vertical plane has been largely developed based on model test for typical deep-water clay deposits with an overlying crust about 1 m in thickness overlying plastic normally consolidated clays. The results and conclusions derived from these tests should not be considered appropriate for other soil conditions.

NOTE The uplift resistance is referred to here as 'suction' although, strictly speaking, under submerged conditions pore pressures normally remain positive.



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Key

1	initial penetration	7	Re-penetration following lift-off
2	uplift	8	ultimate penetration resistance, $Q_u = N_c s_u D$
3	further uplift resisted by suction	9	ultimate suction resistance, Q_{u-suc}
4	suction decays if uplift continues	a	negative reaction (i.e. suction)
5	suction releases with re-penetration	b	normal seabed reaction force, Q
6	further penetration	c	penetration, z

Figure A.20 — Randolph & Quiggin soil model characteristics for different modes [285]

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A.10.4.2 Seabed characterisation

No additional guidance offered.

A.10.4.3 Design for ultimate limit state

No additional guidance offered.

A.10.4.4 Design for fatigue limit state

A.10.4.4.1 General

A.10.4.4.1.1 Elastic spring model

For initial screening evaluations, springs with elastic stiffness can be used. Examples practices where springs with elastic stiffness have been used are given in References [80] and [69]. If the SCR has a lazy wave configuration, the motions at the touchdown point will be reduced versus a standard SCR. The motions can be sufficiently small to minimize cyclic loading effects. Both model tests and analytical models suggest that the soil stiffness, $k = \Delta Q/\Delta z$, normalized by the maximum bearing pressure ($N_c s_u$) for pipe displacements, Δz, of 0,000 3D to 0,002D (where D is pipe diameter) ranges from about 200 to 300. A normalized stiffness $k/(N_c s_u)$ of 200 is therefore recommended. For such small displacements, it is also not likely that the pipe will significantly penetrate and form a trench. A bearing capacity factor (N_c) in the range 5 to 6 should be used to determine the vertical soil stiffness. Many deep-water offshore deposits are characterized by an upper crust with uniform shear strengths (s_u) that can be up to 15 kPa in the upper 1 m to 1,5 m. There is often a reduction in shear strength below the crust followed by an increase following a normally consolidated trend. Due to uncertainties in the pipe penetration, the strength reduction below the crust should not be used to determine the vertical soil stiffness, but the shear strength through the crust should be used. Geotechnical expertise is advised for determining the soil shear strength for the analysis.

For a standard SCR without a lazy wave configuration, the expected motions will be greater and the soil will likely experience more inelastic deformations and reductions in stiffness from cyclic loading. Sectional model tests on a 0,51 m diameter pipe in Gulf of Mexico clay subjected to 1-year winter storm conditions have shown that even for a small number of cycles the average normalised elastic stiffness can be considerably less than the 200 to 300 range for the small displacements noted above [80]. The recommended normalized value for initial screening for a standard SCR is therefore reduced to 100. However, for this case, the pipe will likely penetrate further into the soil and form a trench, because of the inelastic displacements and the effects of cyclic loading. Therefore, a higher bearing capacity factor of 8 is recommended to determine the soil stiffness from the normalized value.

A.10.4.4.1.2 Non-linear spring models

Non-linear models are recommended if the SCR does not meet fatigue requirements with the elastic spring screening model described above.

Two types of non-linear are available. The first developed by Randolph and Quiggin [285] is depicted in Figure A.20. This model has the advantage of directly inputting the shear strength profile rather than interpreting a single representative value. The various input parameters for this model are described in detail in Reference [285]. Although the model does not explicitly account for cyclic loading effects, the re-penetration of the pipe, especially for case where the pipe separates from the soil with lift-off, will provide much lower stiffness values than for case without lift-off. Geotechnical expertise is advised in the selection of the input parameters.

The other non-linear model is based on sectional tests performed on deepwater Gulf of Mexico and Angola seabed soil [79]. These tests cyclically loaded an instrumented sectional riser with a prototype pipe diameter of 0,51 m and length of 100 m. The predicted versus measured fatigue profiles were then compared after 120 cycles of loading. The non-linear soil model was based on shorter (< 1 m) segment tests that were cycled vertically. The tests were performed with a few hundred cycles of loading. Based on the segment data, a hyperbolic soil model was derived which showed very good agreement between the measured and predicted results for the sectional tests. The non-linear soil model was a hyperbolic model, which represents

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an assemblage of points after the soil has degraded for a few hundred cycles. The hyperbolic formulae for the Gulf of Mexico and Angola soils are the following:

$$Q_n = \frac{Z_n}{(A' + B' Z_n)} \tag{A.44}$$

$$A' = \frac{(1-X) Z_{nu}}{Q_{nu}} \tag{A.45}$$

$$B' = \frac{X}{Q_{nu}} \tag{A.46}$$

where

Q_n is the normalized force per unit length of pipe ($Q/(N_c s_u D)$);

Z_n is the normalized displacement (z/D);

X is a curve fitting parameter (approximately 0,85 to 1,0);

Z_{nu} is the normalized displacement at Q_{nu} which is the normalized force at or near the peak action.

For deepwater Gulf of Mexico soil, the parameters X, Z_{nu}, and Q_{nu} that best fit the experimental results were 0,96, 0,10 and 0,35, respectively. For deepwater Angola soil, the best fit parameters were 0,98, 0,10 and 0,125, respectively. N_c is equal to 6,0 for deepwater Golf of Mexico and Angola soils. Z_n increments are typically minimized between the values of 0 and 0,25.

A.10.4.4.2 Selection of s_u below SCR

No additional guidance offered.

A.10.4.4.3 Trenching effects

Field surveys have shown that in some cases trenches a few metres deep can develop [54]. The formation of a trench during the life of an SCR can have an impact on the SCR fatigue life. The trench can be modelled explicitly in the SCR fatigue analyses or correction factors applied depending on the location, direction of loading-vessel position and trench depth. Analysis performed to assess the impact of the trench on SCR fatigue life in Gulf of Mexico are outlined in.[286] Their analyses used a non-linear soil model for a trench with a maximum depth of 5 pipe diameters using three possible trench geometries is summarized in Table A.5 for Gulf of Mexico soil.

Table A.5 — Relative increase in fatigue life for Gulf of Mexico soil and maximum depth of 5D

Platform Motion Position	Average for 3 trench geometries	Range
Near	2,11	1,65 to 2,44
Far	1,03	0,77 to 1,19
Cross	1,05	0,97 to 1,10

NOTE Numbers represent ratio of fatigue life for an SCR with a 5D trench relative to a flat seabed, modelled using a non-linear seabed model (from Reference [286])

Experimental results for a 3D trench indicate the ratio of fatigue life for the trench condition versus a flat seabed is 1,05 [79]. The boundary conditions for these tests (no moment at loading point) are best represented by the 'far' platform motion position, which compares favourably to the average Gulf of Mexico ratio (1,03) shown in Table A.5.

It is difficult to determine the trench depth as it involves complex riser-soil-water interaction processes as well as the vessel motions. If field observations can verify potential trench depths (3D to 5D maximum depth) based on similar vessels and SCRs, Table A.5 can be used to adjust fatigue calculations based on a flat

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seabed. For the 'near' platform motion, the lower bound value of the range is recommended. For the 'far' and 'cross' positions, the average is recommended. If the trench depth cannot be verified, no adjustments to the flat seabed results should be applied.

Some studies have suggested a decrease in fatigue life due to trenching for certain combinations of riser geometry and metocean conditions, while most studies suggest an increase. The effect of trenches on riser fatigue life can be investigated by numerical modelling and/or physical testing for case-specific conditions.

A.10.4.4.4 Cyclic and consolidation effects

Riser cyclic action reduces the soil stiffness and generates excess pore pressure in the seabed. When riser comes to a rest, the excess pore pressures dissipate with time and result in soil consolidation, thus increasing the seabed stiffness. There also can be a case where soil consolidation occurs under prolonged riser cyclic action. The combined process is further complicated by the potential for water entrainment after a pause period with no cyclic loading followed by an event that can cause significant fatigue damage. Model testing or specialized laboratory testing can be carried out to determine soil response for site-specific and design-specific conditions. This matter is still subject of further research.

Either segment or sectional tests can be performed to evaluate the potential effects of cyclic loading on seabed. Segment tests are on short pipe sections and can be performed either under 1-g conditions or in a centrifuge. Sectional tests simulate 100 m plus long prototype sections of the SCR that extend through the touchdown zone [79]. These tests are most practically performed in a geotechnical centrifuge. An advantage of these tests is that they can simulate interactions between the pipe and soil along the length of the section which will help determine the impact of the pipe settlement during pause periods.

For both segment and sectional tests displacement- or load-control can be considered. Expert advice is recommended on selection of the type of loading. However, API RP 2RD[24] indicates the motions near the TDP are displacement controlled. Segment tests should include testing over a range of displacements including those sufficient to cause pipe-soil separation. Sectional tests should be performed mostly at displacement or action levels that cause the most fatigue damage (1/2-year to 2-year events). Larger events can be considered to investigate their impact on cyclic degradation and consolidation [354].

A.10.5 Geotechnical design for jetted conductors and top tension risers

A.10.5.1 General

This subclause provides additional information and guidance pertinent to geotechnical design of conductors for top tension risers.

A.10.5.2 Jetted conductors

A.10.5.2.1 General

The conductor is the well foundation and its main function is to resist the axial and lateral loading actions imposed at the wellhead. In stiff clays and sands conductors are generally grouted into pre-drilled oversized holes or driven in-place. However, a technique analogous to wash boring, called jetting, is a feasible alternative for installing conductors in soft clays that are often encountered in deep water. Having originated in the Gulf of Mexico, conductors are now jetted in most deep-water basins in the world.[179] In recent years, many jetted conductors have successfully been installed. Very little literature has been published regarding jetting design, practice and case histories of failures. References [179] and [2] present design approaches and examples of case histories.

The approach for estimating the short-term axial bearing capacity of a conductor is based on the principle of the weight on bit (WOB) available for jetting and reciprocation to penetrate a conductor [179]. This implies that the immediate capacity is not controlled by the soil conditions, but rather by the available weight on bit and the installation method. Physically, the last soil resistance measured during installation, which is given by the last WOB measured, is equal to the immediate capacity. Maximizing the WOB during jetting is therefore of prime importance. This last WOB should be maximized and should be equal to at least 80 % of the available WOB during jetting. The available WOB will typically be calculated by adding the self-weight of

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the surface conductor, the weight of the wellhead housing, the weight of the drill collars, and the drill ahead tool. Common terms used in conductor jetting operation are:

a) bottom hole assembly (BHA): the portion of the drill string inside the conductor from the landing or running tool to the drill bit;

b) drill-ahead tool: a tool similar to a running tool, except that the drill pipe and BHA assembly can be released from the tool, leaving the tool on the well conductor, allowing continued drilling while the conductor remains in place;

c) drilling assembly: composite of drill pipe and BHA;

d) weight on bit (WOB): the portion of total weight of drilling assembly not supported by the drill rig; the weight of the drilling assembly supported by the soil.

A.10.5.2.2 Short-term axial bearing capacity

The relationship given in 10.5.2.2 for the change in average friction factor along the conductor, $\Delta\alpha_t$, was derived from installation data obtained in the deep water Gulf of Mexico [179]. Using the same approach outlined in Reference [179], $\Delta\alpha_t$ for conductor installations in deep-water Angola seabed was established as presented in Reference [118]. The same approach can be used to develop site-specific $\Delta\alpha_t$ relationship with appropriate calibration and validation against field installation data.

A.10.5.2.3 Long-term axial bearing capacity

The experiments conducted in Reference [361] simulated normally-consolidated to lightly over-consolidated ($1{,}0 \leq OCR < 2$) deep-water Gulf of Mexico seabed conditions. A similar approach can be used to investigate long-term axial bearing capacity of jetted piles in other regions.

A.10.5.3 Soil-structure interaction for well integrity assessment

Response of a top tensioned riser (TTR) system near or below the BOP stack largely depends on the soil behaviour under a specific loading condition. Riser-well-soil interaction analysis is carried out to investigate the following two conditions as part of the TTR system design:

a) strength: the reaction of the riser at the ultimate limit state when the vessel has moved a considerable distance from the mean position;

b) fatigue: the fatigue that occurs within the system as a result of repeated cyclic motions with a range of amplitudes and frequencies.

The first loading scenario can occur when a vessel is drastically shifted from its original/normal operating position as a result of a drift-off or a drive-off event. The former has to do with loss of station keeping caused by an environmental loading (e.g. loop current) whereas the drive-off loading can be intentional (i.e. the vessel is moved to facilitate drilling operations). In either case, the loading can be considered as a slow monotonic condition that occurs within tens of minutes to hours. Because this problem is concerned with the limit state condition of the system, the soil response at both low and ultimate state conditions is important to the overall conductor design assessment.

Given the large differences in the structural stiffness of various components in a TTR system above and below seafloor, it is often difficult to assess whether softer or stiffer estimates of soil response will yield conservative predictions for the design. Stiffer estimates would be more likely to suggest a critical bending moment will occur above seafloor, while softer estimates would suggest more critical moments below seafloor. The situation becomes more complicated for the system components that have a capacity sensitive to combination of axial and bending actions, for example the connection at the lower marine riser package (LMRP).

The fatigue problem is governed by the cyclic actions that occur throughout the life cycle of the riser. These actions can occur from the:

— environmental actions on vessel and top portion of the riser;

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— vortex induced vibrations (VIV) on the riser.

Analyses have shown that the peak loading actions are not necessarily the major contributors to the fatigue damage. Rather smaller loading actions caused by more frequent loading events are responsible for most of the fatigue damage. Therefore, characterization of the soil response at small amplitude displacements is particularly important for the fatigue problem. The *p–y* curves developed for ultimate limit state design of pile foundations for steel jackets subjected to monotonic and cyclic storm or hurricane loading are applicable to the case of well strength analysis. However, they are not suitable for fatigue limit state (FLS) assessment of TTR well systems [1].

The approach for the development of *p-y* curves for FLS assessment of TTR well systems is based on the degraded soil secant stiffness (unload-reload stiffness) at the steady-state condition in sands and clays. Results from model tests have indicated that damping in clays can play a role in fatigue damage. As such, *p-y* models have been developed to cover a range of complexity from fatigue *p-y* curves only to fatigue *p-y* curves coupled with soil damping (modelled by dashpot). Fatigue *p-y* curves without soil damping are typically used for base case level studies (initial analysis) that are carried out first. If fatigue life does not pass the acceptance criterion, more refined analysis can be carried out, which includes hysteretic soil damping and/or obtaining more geotechnical information to increase accuracy of soil parameters. For complex situations and/or where site-specific soil samples are available, laboratory testing is recommended to develop fatigue *p-y* curves. An example of the apparatus and approach is given in References [1] and [357].

A.10.5.4 Geotechnical input to well strength assessment

No additional guidance is offered.

A.10.5.5 Geotechnical input to well fatigue assessment

A.10.5.5.1 General

Context on the development of the soil-structure interaction for well fatigue analysis, which form the basis of the guidance outlined in this subclause are provided in References [356], [359] and [360]. The approach has been validated through field monitoring programs. Reference [246] presents specifics of a field monitoring program of an instrumented well installed in the North Sea with the seabed comprising layered clays and sands. Reference [296] compares the stresses measured in the field during drilling operations to those obtained numerically from a full three-dimensional (3D) finite element (FE) analyses model of the blowout preventer (BOP), wellhead (WH), conductor, and surface casing versus the field measured data obtained. The numerical analyses used the backbone *p-y* response outlined in earlier editions of this document and API RP2GEO[21] and the spring-only method presented in References [356], [359] and [360]. Sensitivity studies were also performed. They concluded that the spring-only method yields good prediction of the stresses measured in the field in the conductor and the surface casing.

Reference [189] presented field measurements made at the lower marine riser package (LMRP) location from monitoring a well installed in normally consolidated to lightly over-consolidated clays in deep-water Gulf of Mexico and evaluated performance of the spring-only method presented in References [356], [359] and [360] through 3D FE numerical modelling and fatigue analysis. The numerical model consisted of the well system below the LMRP to 50 m below the seafloor and simulated sea states observed during the drilling operation. The measured motions and fatigue damage obtained from the loggers were compared to those estimated from predictive methods using the spring-only method and it was concluded that the spring-only model performs satisfactorily in predicting the deformations (displacements and rotations) measured at the top of the LMRP; hence, providing a more accurate prediction of the system (LMRP, wellhead and casings) response and thereby, its fatigue damage.

Reference [137] presented a detailed well monitoring program with improvements made to the algorithm and measurement accuracy technique for the purpose of well fatigue assessment in layered sand and clay seabed. The sand layers ranged from loose to very dense and the clay strata consisted of soft to very stiff at depth. Reference [236] conducted fatigue analysis using the backbone *p-y* outline in earlier editions of this document and API RP2GEO and the spring-only method outlined in References [359] and [360]. They concluded that the predictions made with the spring-only method provided BOP response similar to those observed in the field. However, the use of the API model significantly overestimated ‘measured’ conductor

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fatigue life above the seafloor. Ultimately, the spring-only method was considered more suitable for wellhead fatigue assessment than the backbone *p-y* curve in earlier editions of this document and API RP2GEO.

The guidance given specifically states that the soil-structure interaction in the fatigue analysis model should not be done with simulating the vessel/rig in offset position. This is solely to avoid erroneous numerical results arising from user errors or limitations in the existing tools and software packages. The issue of vessel offset and soil response was investigated for a wide range of motions and initial offsets in the centrifuge tests presented in References [359] and [360]. The soil response was found to be independent of the initial offset positions (i.e. the soil degradation and resistance in both with and without offset were found to be the same). Shape and characteristics of fatigue *p-y* differ from those of monotonic *p-y*. The stresses needed for input to well fatigue analysis is typically obtained through numerical analysis with the model of the well and foundation soil (represented by *p-y* springs) combined. If the intention is to specifically evaluate fatigue damage from a situation where the vessel/rig is in offset, separate simulations will be required for each offset motion and fatigue motion phases with the appropriate *p-y* used to model the soil response for each loading condition (i.e. monotonic *p-y* for offset loading and fatigue *p-y* for fatigue loading).

A.10.5.5.2 Clays

A.10.5.5.2.1 Spring-only method

No additional guidance is offered.

A.10.5.5.2.2 Spring-dashpot method

Soil–structure interaction analysis may include material damping (hysteretic) in the soils and structural materials as well as the foundation radiation damping. The energy dissipated during each load-unload loop is termed hysteretic damping. Radiation damping (also known as the geometric damping or geometric attenuation) differs from material damping in which elastic energy is dissipated by viscous, hysteretic or other mechanisms. Estimating each type of damping in a system is often difficult due to complex interplay of material damping and radiation damping in the dynamic solution. The effects can be approximated in a linear analysis by introducing equivalent viscous damping with energy dissipation equal to that dissipated by hysteresis, an example of which was demonstrated in Reference [360].

A.10.5.5.3 Sands

No additional guidance is offered.

A.10.5.6 Geotechnical considerations in conductor driving analysis

A.10.5.6.1 Conductor pipe installation

The process involving conductor pipe installation is similar to pile driving. Pile installation considerations are described in 8.7.11. Some additional guidance specific to conductors are provided in this subclause.

A.10.5.6.2 Minimum wall thickness

A.10.5.6.2.1 General

The minimum wall thickness requirements of ISO 19902 for piles can be insufficient to mitigate the risk of local over-stressing for conductors. For instance, wall thickness values as small as 14 mm (approximately 0,55 in) to 16 mm (approximately 0,63 in) are generally not acceptable for 0,762 m (30 in) to 0,914 m (36 in) diameter conductor pipes.

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A.10.5.6.2.2 During driving

The following is recommended for implementation during conductor pipe driving:

a) The allowable maximum driving stresses as estimated from wave formula analyses should possibly be decreased to less than 80 % to 90 % of the yield strength in the following cases (often associated with larger uncertainties):

b) use of steam or diesel hammers instead of hydraulic hammers;

c) maximum driving stresses not predicted at or close to pipe head (e.g. above the driving shoe or close to the pile tip);

d) presence of hard driveable layers (with noticeable tip reflection ratio, i.e. ratio between reflected and incident stress wave).

e) The wave formula analyses undertaken to predict the driving stresses should take into account soil variability, particularly to simulate the effect of local harder layers/pockets at pile tip.

f) For foundation piles, the total stress during driving should be estimated at any point along the conductor pipe and should include the driving stress, the weight of the hammer assembly and the bending stresses due to inclination of the conductor pipe/hammer as well as due to conductor pipe deviation.

A.10.5.6.3 Driving fatigue

Driving fatigue assessment for conductor pipes follows the same guiding rules as for driven piles, particularly if the sections are welded.

If the conductor pipe segments are connected by means of mechanical connectors, larger stress concentration factors (SCF) can possibly be applicable. The applicable SCF should be defined by the manufacturer.

A.10.5.6.4 Conductor tip damage and buckling

A.10.5.6.4.1 General

The potential to damage the tip of the conductor pipe as well as to propagate any potential damage during driving should be verified. The risk is considered highest in the following conditions:

a) in presence of very dense sands, cemented soils, very weak rock or boulders;

b) for the case of drive-drill-drive installation sequences, after drilling when driving is resumed (without soil support on the inside of the conductor pipe);

c) at shallow depth or when the shaft resistance is small compared to the end bearing resistance.

Although the results of wave formula analyses give some insight into the potential of pile tip overstressing during driving, this type of analysis remains a 1D analysis that does not properly capture the potential for localized buckling.

A.10.5.6.4.2 Damage during driving

The dynamic point loading actions acting locally on the conductor pipe tip during driving can be evaluated and compared to the loading actions that can cause damage. This requires some assumptions to be taken regarding the proportion of the conductor pipe annulus hitting a harder layer or an obstacle, such as a boulder, and the static resistance to be overcome (for a boulder, this will be the force required to either push the boulder into the surrounding soil or split the boulder, whichever is less). Wave equation analyses simulating the selected scenarios either in a 1D traditional wave equation model or a dynamic 3D FE model

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will then be performed. An example of wave equation model of a large diameter pile hitting a boulder is presented in Reference [164], which provides:

a) a stiffness criterion based on the ratio conductor pipe diameter/wall thickness and the elastic moduli of soil and pile; this allows verifying whether the soil elastic response is stiffer than the one of the pile. Otherwise, any inwards pile deformations will spring back and recover elastically;

b) a strength criterion as a function of the wall thickness and the soil strength; there should be enough pressure from the soil acting on the initial small damaged area of the pile tip to cause further yielding of the steel.

If the high-level evaluation indicates a potential for damage/imperfection propagation, a numerical model simulating pile extrusion, such as presented in Reference [117], may be implemented.

A.10.5.6.5 Installation with inclined shoes

To reduce the risk of interaction of closely-spaced conductor pipes, the peripheral conductor piles can be deviated away from the group. Installing an inclined (bent) driving shoe will allow creating a tendency for the pile to deviate in a particular direction. Predicting the path followed by the conductor pipe during driving is necessary so that the lateral deviation can be controlled.

As conventional beam-column theory will not be applicable in this case. Reference [315] describes a simple finite difference model to predict the path of a bent shoe during driving, extending the equations presented in Reference [265] for deviation of curved/bent piles. The parameters acting on the deviation of the pile are the axial tip and side resistance of the pile, the lateral soil resistance and the length and inclination of the driving shoe, with the latter three parameters having most effect. [113][141]

A.10.6 Foundation design for riser towers

A.10.6.1 General

No further design guidance is offered.

A.10.6.2 Foundation options

No further design guidance is offered.

A.10.6.3 Loading actions and safety factor

A.10.6.3.1 Loading actions

No further design guidance is offered.

A.10.6.3.2 Recommended safety factors

The safety factors recommended by ISO 19901-7 and API RP2T [1] for driven piles and gravity base anchors and by API RP2SK [22] for suction piles may be increased because of the following.

a) The API RP2SK safety factors were developed with no consideration of permanent uplift loads on suction caissons. Additional information on permanent uplift loads is provided in A.10.6.

b) API RP2T does not address the case of gravity loads explicitly and 10.3.3 of API RP2T, 3rd Edition, July 2010, states that:

   For axial pile design where the weight of the foundation system is less than approximately 10 % of the ultimate axial capacity, the underwater weight of the foundation system may be subtracted from the applied loads in determining the safety factor of the foundations. For other weight-dominated systems, the foundation system weight should be added to the resistance side of the formula.

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A.10.6.4 Design challenges

A.10.6.4.1 General principles

Design of riser tower foundation subjected to uplift action addresses the following aspects::

a) penetration and retrieval;

b) holding capacity including long-term uplift capacity;

c) long-term displacement;

d) soil reactions to be used for the structural design.

The capacity should be checked both for the permanent actions and for the sum of permanent and cyclic actions.

The undrained shear strength for the permanent actions may be reduced to account for long term creep effects (see examples in References [220], [212] and [324]). For sustained loading action due to loop currents that are expected to last several days up to two weeks, it is recommended to apply a strength reduction factor of 20 % on the undrained shear strength. The same 20 % reduction is recommended for long-term permanent actions.

The effect of pore pressure redistribution and swelling should also be estimated, since this can lead to reduction in effective stresses and undrained shear strengths, and hence reduce the capacity under transient wave loading action [212]. In cases where the permanent action acts for many months or more, it is recommended to evaluate drained or partially-drained conditions to determine whether full base suction can be maintained. References [1] and [84] show that the capacity of a typical suction caisson of depth to diameter ratio between 4 to 6 under a sustained loading action in normally to slightly over-consolidated clay can be only about 70 % of the capacity for a rapid loading action. This reduction, however, only considers the loss in reverse end bearing (REB) (base suction). An additional reduction in undrained shear strength of 20 %, is advisable. Therefore, the reduction in base suction should include both the reduction in shear strength and the reduction from drainage for base suction. Numerical analyses with specific geometries and soil conditions will provide better estimates of the potential reductions from drainage long term creep effects. Unless demonstrated by physical and/or field testing, a maximum of 50 % REB is recommended for inclusion in the capacity assessment, and only for plastic clays under short-term loads.

Drainage and pore pressure redistribution can also influence the undrained shear strength under cyclic loading actions, but the shear strength for the sum of permanent and cyclic loading actions will be higher than for the permanent loading actions due to rate effects (see example in Reference [9]).

The potential of gapping for suction caissons along the wall above stiffeners or sections with increased wall thickness should be evaluated. In cases with ring stiffeners, it is necessary to evaluate the potential of trapped water between ring stiffeners (see example in Reference [12]). If gapping or trapped water is possible, the effect that such gaps will have on the drainage path and resistance due to lack of contact along the wall should be evaluated.

The capacity of suction caissons can depend on passive underpressure inside the caisson that contributes to REB. If REB is relied upon, proper sealing can be critical, especially for the part of the underpressure generated by long period environmental actions, such as loop currents. If the anchor top valve seals cannot be guaranteed, either a back-up cap behind the valves or a monitoring program should be considered to assure the desired integrity over the lifetime of the suction caissons. Seepage simulations through an unsealed valve can be conducted to evaluate the effect of a leak on bearing capacity. If proper sealing is not ensured, the suction caissons should be designed to resist sustained uplift action without taking REB into account, or with a reduced REB taken into account allowing for the effect of leakage. If REB is not taken into account, the capacity is calculated from the skin friction of the inner and outer walls with appropriate adhesion factors. The increase of capacity with time and the REB capacity with closed versus open top are described in A.11.5.2.2.4 and A.11.5.2.2.7.

Sustained permanent tension and creep fatigue for long periods of several years or more was further assessed in Reference [190], which showed the drainage effect in relation to phenomenon known as stress

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relaxation around the pile perimeter. The effect of drainage can result in increased soil strength and foundation capacity. The tests also showed that the magnitudes of time effects on soil strength increase with increasing water content and plasticity and presented governing relationships for creep behaviour.

A.10.6.4.2 Assessment of effect of sand layers on suction anchors

A simple seepage check on drawdown of a suction pile with the tip in sand layer is recommended. The calculated flow should ensure that the vertical displacement of the pile for the duration of the peak dynamic action remains small and within the elastic range of soil plug load-displacement response.

As illustrated in Figure A.21, the discharged water flow, Q, can be calculated by:

$$Q = \frac{2\pi kH}{Ln\left(\frac{r_1}{r_2}\right)}\frac{p}{\gamma_w} \tag{A.47}$$

where

k is the coefficient of permeability of sand layer;

H is the sand layer thickness;

p is the applied underpressure;

r_1 is the radius of the suction anchor;

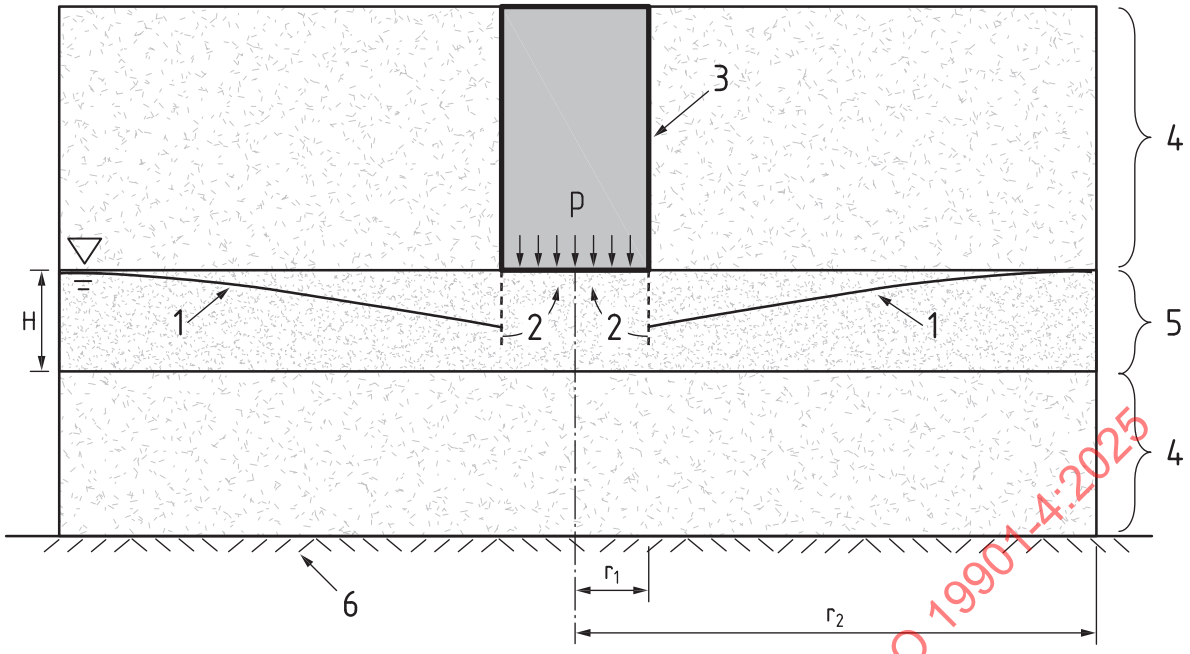
r_2 is the radius of influence soil under suction, which is typically about 5 times the anchor radius;

γ_w is the water unit weight.

Using Formula (A.47), the flow across the suction pile for the duration of the peak dynamic action can be calculated, from which the potential uplift of the suction plug can be calculated. This will determine if the vertical displacement is sufficiently small to ensure that suction anchor stability remains satisfied.

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Key

1	drawdown	H	sand layer thickness
2	flow, Q	p	applied underpressure
3	suction pile	r_1	radius of suction anchor
4	clay layer	r_2	radius of influence soil under suction. Typically, about 5 times the anchor radius
5	sand layer		
6	top of impervious layer		

Figure A.21 — Illustration of water flow in sand layer below a suction pile under tension

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A.10.6.5 Inspection and monitoring

No further guidance is provided

A.10.7 Offshore pipelines and flowlines

A.10.7.1 Geotechnical pipe-soil interaction (PSI) analysis

A.10.7.1.1 General

Pipeline design, buried and unburied, considers ultimate and fatigue limit states related to the stresses in the pipeline and the movements of the associated end connections, including sections that transition into a catenary riser or lift off the seabed into a termination assembly.

Geotechnical analysis leads to estimates of pipeline embedment and the force–displacement response in the axial, and lateral, and vertical directions. PSI model is typically developed in stages. The simplest axial and lateral PSI model comprises limiting friction factors for each direction of movement, while more complex models capture features, such as the accumulation of soil berms beside the pipe during cyclic motion. Development of the PSI model can be completed after the desk study or the preliminary stage depending on soil conditions, load conditions, or regional experience. At the preliminary and detailed stages, the complexity of the PSI model should be designed to address:

a) level of uncertainty in soil parameters;

b) likelihood of reduction in PSI uncertainty;

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c) differences in project-specific design conditions and those used to develop the recommended practices for unburied and buried pipelines;

d) methodology to combine uncertainties in soil properties, pipe properties, and model.

Complex models typically do not reduce overall uncertainty in PSI if input soil and pipeline properties are not well-defined. Model uncertainty (and some aspects of soil parameter uncertainty) can be reduced through physical model testing or finite element modelling. Complex analysis can offer insight if the project conditions are outside the previous experience.

Detailed guidance on methods developing geotechnical input to PSI analysis for buried and unburied pipelines are provided in DNV RP F110 and DNV RP F114, respectively. DNV RP F114 is based on SAFEBUCK Guidelines.[297] The design guidance outlined in these two documents address design situations along the main length of a pipeline and not the following situations:

— close to where pipelines terminate at a point of fixity;

— along very short pipelines (e.g. spools);

— where pipelines are supported on crossings or by buckle management structures.

A.10.7.1.2 Buried pipelines and flowlines

No further guidance is offered.

A.10.7.1.3 Unburied pipelines and flowlines

No further guidance is offered.

A.10.7.1.4 Actions on pipelines

A.10.7.1.4.1 General

PSI analysis should address all applicable actions on the pipeline. This involves determining effects of rate, duration and frequency of loading, and therefore the resulting soil response.

The actions and motions imposed during laying govern the pipeline embedment at the start of operation and initial in-service performance. The actions on a pipeline after laying include hydrodynamic or thermal loading, internal and external pressure and changes in self-weight from changes in contents, including rapid cycles of changing weight from slug flow. An element of pipeline can also be subjected to actions from adjacent sections of pipeline, including a connected catenary riser, and from end connections.

Accumulation of axial movements (pipeline 'walking') can occur due to operating cycles of internal pressure and temperature, which can affect external connections. Where walking leads to increased compressive axial forces, a risk of lateral buckling of the pipeline arises. A pipeline can also be susceptible to external loading from debris flows and turbidity currents that arise from submarine slides and snag or impact loading from foreign objects.

A.10.7.1.4.2 Determination of PSI input parameters

During the preliminary stage, the following parameters should be determined for the range of normal stresses anticipated to be exerted by pipeline on the seabed:

a) drained residual pipe-soil interface friction angle; friction angle can be measured using interface shear box or tilt table;

b) ratio for the normally consolidated interface shear resistance versus the consolidation vertical stress, $\left(s_{u,in,res} / \sigma'_{vo}\right)_{nc}$, for the estimated range of pipe roughness and weight; this ratio is stress dependent; it can be measured using interface shear box or shallow penetrometers;

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c) transition rate from undrained to drained interface resistance through cycles of shearing separated by rest periods, or during continuous shearing;

d) coefficient of consolidation; coefficient of consolidation can be measured from consolidation stage of shear box test, from site investigation data or other consolidation tests.

During the detailed stage, the following activities may be performed:

— low stress interface testing for design ranges of pipe weight, coating, and velocity;

— model testing.

Model testing can include 1 g axial testing in test bed and centrifuge lateral testing. Activities are performed based on value opportunities and risks associated with pipeline design and PSI inputs, including whether the pipe and/or soil parameters are outside of available data and experience.

A.10.7.1.4.3 Drained and undrained behaviour during pipe-soil interaction

In fine-grained sediments, pipeline laying is usually an undrained process. Dissipation of the lay-induced excess pore pressures typically takes days or weeks. Lateral pipeline movements in response to hydrodynamic action or thermal expansion generally involve undrained deformation, although consolidation between events can cause disturbed soil to regain strength. Axial pipeline movements can be drained or undrained in fine-grained soil since the relevant drainage distances are shorter than for lateral movement.

In coarse-grained sediments, pipeline installation and operation will generally occur under fully drained conditions. In intermediate soils, such as silts, silty sands and silty clays, it can be necessary to consider both drained and undrained conditions.

In a design analysis, the anticipated durations of axial and lateral pipeline movement or loading should be compared with the relevant rates of consolidation to establish whether drained, undrained or partially-drained conditions will prevail. It can be necessary to perform analysis of both short and long duration pipe movements relative to the soil consolidation to bracket the expected range of PSI parameters.

Approximate durations are provided in Reference [149] for consolidation under vertical or lateral pipeline loading, and in Reference [353] for axial pipeline movement. Key values of dimensionless time, T_{50}, for 50 % dissipation of pore pressure are provided in Table A.8. The dimensional time, t_{50}, is calculated from T_{50} using the coefficient of consolidation, c_v, and pipe diameter, D. This time can be compared to the duration of a loading or movement event, t_{event}, to estimate the relevant drainage condition(s). Since the transition between undrained and drained conditions spans approximately a factor 100 in time, if $t_{event} > 10t_{50}$, then drained conditions can be assumed, and if $t_{event} < t_{50}/10$, then undrained conditions can be assumed [149], [353]. For the intermediate range, both undrained and drained conditions can be considered, or a more detailed analysis can be performed to justify a narrower range of conditions.

Table A.8 — Consolidation periods during pipe-soil interactions (based on References [149] and [353])

Dimensionless time, $T_{50} = c_v t_{50}/D$	T_{50}
Vertical loading	0,1
Lateral loading	0,02
Axial loading	0,005

A.10.7.1.5 Axial soil resistance

The parameter $R_{nc} = (s_{u\text{-}int\text{-}res}/\sigma'_{vo})_{nc}$ is the normally-consolidated interface undrained strength ratio, at large displacements. The index parameter, m, is equivalent to the plastic volumetric strain ratio in Cam clay or the SHANSEP over-consolidation index, and typically ranges between 0,5 and 1. The interface model parameters should be measured in low stress interface shear box tests, as described in Reference [44], or via shallow penetrometer devices (see Reference [300]). The drained parameter, δ, can alternatively be measured via tilt table tests (see Reference [248]).

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n the absense of site-specific testing, the parameters presented in Table A.9 may be used for initiation design purposes. Obtaining site-specific data for design purposes is recommended.

Table A.9 — Database values of pipe-soil interface properties (from Reference [335])

	Normally-consolidated residual interface undrained shear strength ratio $(s_{u,in,res} / \sigma'_{vo})_{nc}$	Drained interface strength ratio $\tan(\delta_{res})$
Low estimate (P10) value	0,22	0,34
Best estimate: median (P50) value	0,33	0,50
High estimate (P90) value	0,46	0,89

A.10.7.1.6 Performance evaluation stage

Performance evaluation stage is also referred to as the "back-analysis" stage and involves two aspects: post-installation survey and monitoring, analysis.

a) Survey and monitoring

Post-installation surveys should include:

1) a pre-flooding survey with measurements of:
   i) out of straightness;
   ii) embedment;
   iii) end displacements, through general visual inspection;
2) a post-hydrotest survey with measurements of:
   i) out of straightness;
   ii) end displacements, through general visual inspection;
3) a survey immediately post-start up with measurements of:
   i) out of straightness;
   ii) end displacement, through general visual inspection;
4) in-service surveys with measurements of:
   i) end expansions;
   ii) embedment;
   iii) buckle monitoring;
   iv) pipeline anchor performance, if any;
   v) route curve stability.

Post-lay pre-flooding survey provides reference datum for subsequent surveys. In-service surveys during operations can be performed annually or at longer intervals, depending on observed response and assessed structural integrity.

b) Analysis

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Pipe embedment and seabed stiffness back analysis should include:

1) statistical assessment of as-installed embedment, using a minimum of 5-points (near and far on both sides plus top of pipe); near refers to the soil adjacent to the pipe, including soil heave due to pipe embedment; far represents the points either side of the pipe distant from soil heave areas;

2) continuous cross-profiles; continuous cross-profiles can be obtained with high accuracy swathe survey to provide high-resolution digital terrain mapping.

Axial response back analysis should include:

1) matching of end expansions and walking rates, if any, by parametric variation of axial pipe-soil interaction model parameters;

2) evaluation of end expansions and walking rates sensitivity to PSI load deflection response including:

   i) peak break-out resistance value;

   ii) peak break-out mobilisation displacement;

   iii) displacement elastic slip;

3) effect of feed in and feed out to buckles and route curve pull out.

Lateral response back analysis should include:

1) matching of buckles (shapes, number and profiles) by parametric variation of lateral PSI models; back analysis uses the continuous cross-profiles;

2) evaluation of lateral buckling sensitivity to the axial model; there can be more than one combination of axial and lateral responses that match the buckle shapes.

A.10.7.1.7 Sediment transport and scour

If the seabed is mobile, scour beneath a pipeline can form a free span. The suspended sections of the pipeline can sag into the scour hole, or sufficient length of the pipeline can be undermined to cause the remaining span shoulders to fail in bearing due to the pipe weight, resulting in lowering of the pipeline relative to the seafloor. Sediment can also be deposited around the pipe. A consequence of these processes is that the pipeline embedment can evolve with time and free spans can form and remain stable or can evolve, migrate or be eliminated

Spans can be subjected to vortex-induced vibration leading to fatigue, greater exposure to submarine slides and a change in the hydrodynamic stability (Scour and sediment transport can also affect the pipeline behaviour at engineered buckles due to the evolving embedment and resulting changes in lateral resistance.

Scheduled monitoring of scour and sediment transport can form part of a design strategy for limit states that evolve gradually over time such that intervention is possible if required (e.g. fatigue and pipeline walking). Observations of existing nearby pipelines can be useful to quantify the likely behaviour of new pipelines.

Laboratory tests in erosion functional testing apparatus can be carried out on site-specific soil samples to evaluate the erosion potential. Techniques for assessing the condition of pipeline free spans are outlined in Reference [102]. Mitigation methods, such as rock dumping and other stabilisation methods, can be used to rectify unacceptable free spans.

A.10.7.1.8 Damping from pipeline-seabed interaction in free span analysis

Pipeline free spans or slugging-related issues for a pipeline bridging over a sleeper require assessment for fatigue potential. The stiffness and damping provided by the soil supports can have a significant influence on the fatigue rate. DNVGL RP F105 provides practical advice on the assessment of free span behaviour and recommends soil damping ratio based on soil classification (clay – for three consistencies from very soft to hard; sand – loose, medium or dense) and free span length. Damping ratios from 0,5 % to 3 % are specified, increasing with soil stiffness and reducing with span length.

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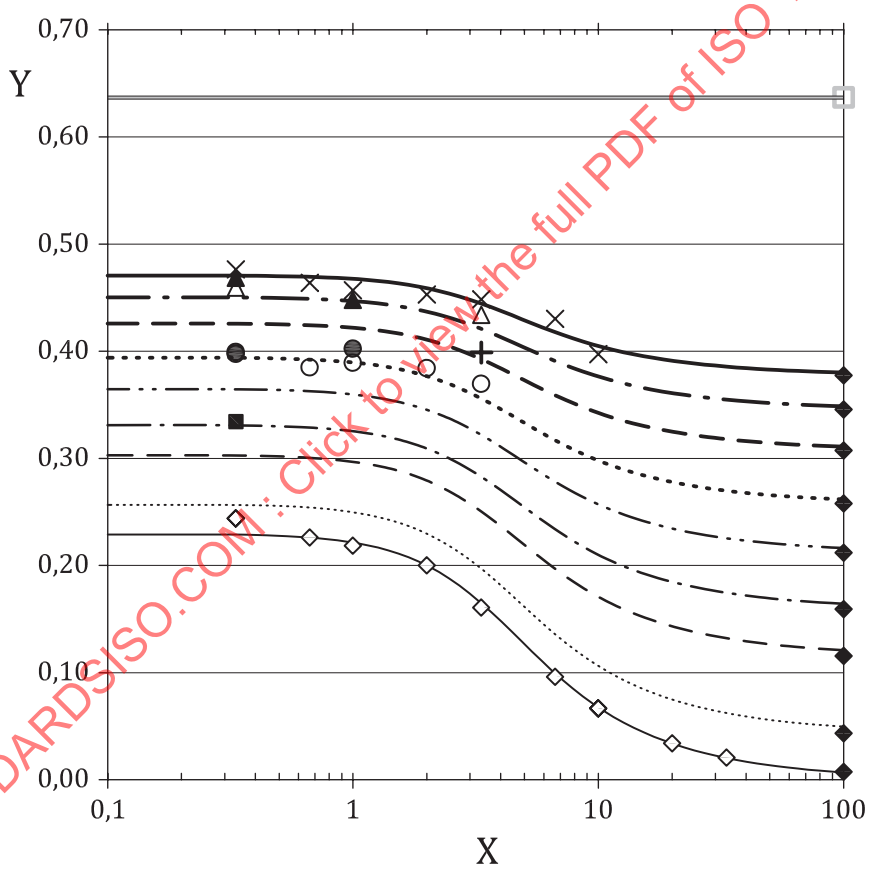
In contrast to these recommended levels of damping, recent work discussed in Reference [323] has shown higher damping levels for a partially embedded pipe on the seafloor in very soft clay. Their results were obtained by modelling a 0,3 m (1 ft.) diameter pipe embedded 0,25D, varying the amplitudes and frequencies of motion to determine both hysteretic and radiation damping ratios. The analyses considered both types of damping and a combination rule proposed to determine the sum of both components (see Figure A.22).

The results indicated greater levels of damping versus what is usually considered in design, including the advice in DNVGL RP F105 For example, for loading periods from 0,1 s to 1 s (1 Hz to 10 Hz), the damping ratio from radiation damping alone was about 0,2. For very long loading periods (100 s), where radiation damping is negligible, the damping ratio ranged from about 0,22 to 0,38 for displacements ±0,006D to ±0,04D.

Preliminary sensitivity analyses have been performed to assess the potential impact of these high damping ratios on the fatigue of a pipeline span. The pipe diameter considered was 0,46 m (18 in) while the span length was about 50 m. The results show significantly reduced displacements at the shoulders and mid-point of the span for higher damping levels. In addition, about a 15 % to 25 % reduction in the stress range at the mid-span was observed for damping ratios greater than 20 %. This level of stress reduction will increase fatigue life or allow longer acceptable span lengths.

Analyses similar to those performed by Reference [323] can be used to define appropriate damping ratios for other conditions.

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Key

X	normalized period, T/T*	+	FEA (exp), 0,02
Y	damping ratio (combined), D	○	FEA (exp), 0,01
□	theoretical maximum material hysteric damping	●	FEA (std), 0,01
═══	maximum combined damping	■	FEA (std), 0,004
———	comb. Damp. Rule, SA displ./dia.=0,04	◆	FEA (std), material hysteric damping only
— · —	0,03	D	$D_{hyst} + a \times D_{rad}$
– – –	0,02	D_{hyst}	material hysteretic damping ratio

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.....	0,01	a	fraction of energy not dissipated by material hysteresis
— · · —	0,006	D_{rad}	radiation damping
— · —	0,004	Displ./dia	displacement / pipe diameter
– — –	0,003	FEA	finite element analysis
.....	0,002	EXP	explicit solution
———	0,000 2 or less (radiation damping only)	SA	single amplitude
◇	FEA (exp) radiation damping only	STD	standard (implicit) solution
×	FEA (exp), single amplitude displacement/pipe diameter =0,04	T	period
△	FEA (exp), 0,03	T*	100 × pipe diameter / shear wave velocity
▲	FEA (std), 0,03		

Figure A.22 — Damping ratios for 0,3 m diameter pipe resting on seafloor (from Reference [323])

A.10.7.2 Submarine slides and density flows: simulation and pipeline impact analysis

A.10.7.2.1 General

This subclause sets out a recommended procedure for the simulation of submarine slides and density flows (both referred to as 'flows' here) and the resulting pipeline impact analysis following the steps listed in 10.7.2.

A.10.7.2.2 Rheology characterization

Characterization of flow materials is typically carried out using a rheometer. Flow characteristics of soils can be described using non-Newtonian rheological models, such as the Herschel-Bulkley model, which can be expressed using either fluid mechanics (Formula (A.48)) or geotechnical notation (Formula (A.49)), as:

$$\tau = \tau_y + K\dot{\gamma}^n \tag{A.48}$$

$$s_{u,op} = s_{u0}\left[1+\eta\left(\frac{\dot{\gamma}}{\dot{\gamma}_{ref}}\right)^{\beta}\right] \tag{A.49}$$

where

τ_y (or s_{u0}) is the yield stress or strength at very low or zero strain rate γ,

K is a (dimensional) 'consistency' parameter,

η is a dimensionless viscosity parameter, while n (=β) express the dependency of strength on strain rate.

τ and $s_{u,op}$ both represent the mobilised or apparent shear stress (strength). The alternative forms of Formulae (A.48) and (A.49) are fully interchangeable and achieve the same objective of a rate dependent mobilised 'strength' of the flow material. Selecting the reference strain rate $\dot{\gamma}_{ref}$ to match the strain rate in conventional geotechnical element tests allows data from different tests to be combined.

The parameters in Formulae (A.48) and (A.49) should be estimated through laboratory testing that covers the relevant range of strength, shear strain rate and moisture content. Using a vane-in-cup rheometer in accordance to ASTM D2196 [27] is recommended for high strain rate testing. It is recommended to conduct the rheology tests at soil temperatures representing the site conditions for a range of moisture contents.

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A.10.7.2.3 Flow simulation

Simulation of the run-out process of a submarine slide or flow can be performed numerically using techniques of varying complexity. It is common for depth-averaged Eulerian models to be used, with the slide running out over a 3D bathymetry. More complex analyses, using computational fluid dynamics or a large deformation finite element method, can also be used to simulate slide behaviour.

Analyses of submarine slide and flow behaviour are often calibrated on a site-by-site basis, using relic slides that are identified from the bathymetry, geophysical data and sample dating.

A.10.7.2.4 Mobilised shear strength of slide / flow material

In assessing the impact force on a pipeline, the relevant flow velocity v is the *relative* velocity between the moving density flow (or soil) and the pipeline. This is important for conditions where a flow might engulf a pipeline and carry it forward until such point where equilibrium is achieved between the forces exerted on the pipeline by the decelerating flow and the structural forces induced in the deformed pipeline. Appropriate values of operational shear strength should be assessed, taking account of the strain rate dependency of soil stiffness and shear strength. The nominal shear strain rate in the soil flowing around the pipeline, $\dot{\gamma}$, is expressed as:

$$\dot{\gamma} = f\frac{v}{D} \tag{A.50}$$

Use $f = 1$ when calculating $\dot{\gamma}$ to find τ and Re_{nN} to use in the fluids-based approach (Formula (A.52)). Use $f = 0{,}9$ when calculating $s_{u\text{-}op}$ and Re_{nN} to use in the geotechnical-fluid hybrid approach (Formula (A.55)). This difference in definitions of nominal shear strain rate originates from the calibration of each approach.

A.10.7.2.5 Impact forces on pipelines

The forces arising from pipeline-flow interaction, including situations at the onset of a slide, can be estimated analytically using the estimated properties of the flow material. When a flow impacts a pipeline, the force that is applied to the pipeline depends on (i) the inertia (mass density) of the material, which is deflected around the pipe, and (ii) the strength (or mobilised shear stress) within the material, which is deformed as it flows around the pipe. The relative influence of the material inertia and strength depends on a non-Newtonian Reynolds number:

$$Re_{nN} = \frac{\rho v^2}{\tau} \text{ or } \frac{\rho v^2}{s_{u-op}} \tag{A.51}$$

Approaches to estimate impact forces have initially developed from fluid dynamics-based principles where the force is assumed to be generated by fluid drag, with velocity dependent drag coefficients. For values of Re_{nN} greater than 2, drag forces dominate and models based on fluid mechanics alone, with velocity dependent drag coefficients, can be used [357]. Subsequently, a combined geotechnical-fluid approach (References [283][298] and [215]) was developed using the data presented in Reference [357], which is also suitable at low Re_{nN} where the strength of the debris can dominate the impact force. Both methods are outlined in this subclause. Impact forces can be calculated using both methods, which should yield similar results. In case of a discrepancy, reference should be made to the experimental and numerical dataset given in Reference [357] that underpins the two approaches, and other relevant data, in order to compare the suitability of each calculation method.

The force also depends on the angle of attack of the flow relative to the alignment of the pipeline, θ, where $\theta = 90°$ for flow perpendicular to the pipe axis, and $\theta = 0°$ for flow parallel to the pipeline.

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One of the following expressions for impact forces should be used. Both can be evaluated for comparison, and if there is an unacceptable discrepancy between the methods, then reference should be made to the underlying data to evaluate which approach is most suitable.

a) Fluids-based approach

For the two extremes of attack angle, the pipeline force per unit length, F, is:

$$F_{\theta=90} = \left[C_{D,90}\left(\frac{1}{2}\rho v^2\right)\right]\mathrm{D} \qquad \text{and} \qquad F_{\theta=0} = C_{D,0}\left(\frac{1}{2}\rho v^2\right)D \tag{A.52}$$

where v is the depth-averaged flow velocity over the height of the pipe. For fully-engulfed flow around a pipe, the velocity-dependent drag factors are given by [357]:

$$C_{D,90} = 1{,}4 + 17{,}5\,Re_{nN}^{-1{,}25} \qquad \text{and} \qquad C_{D,0} = 0{,}08 + 9{,}2\,Re_{nN}^{-1{,}1} \tag{A.53}$$

For a laid-on-seafloor pipeline [357]:

$$C_{D,90} = 1{,}25 + 11{,}0\,Re_{nN}^{-1{,}15} \tag{A.54}$$

b) Geotechnical-fluid hybrid approach

In the geotechnical-fluid hybrid approach, the impact force for perpendicular attack comprises two terms. The first term relates to inertial loading, with a drag factor, $C_{\mathrm{D}\text{-}90}$, and the second relates to strength loading, with a bearing factor, $N_{p\text{-}90}$. The axial impact force includes an adhesion factor, f_a:

$$F_{\theta=90} = \left[C_{D,90}\left(\frac{1}{2}\rho v^2\right) + N_{p-90}s_{\mathrm{u,op}}\right]\mathrm{D} \qquad \text{and} \qquad F_{\theta=0} = f_{a,0}s_{\mathrm{u,op}}\pi D \tag{A.55}$$

For fully engulfed flow around a pipe, $C_{\mathrm{D},90} = 0{,}6$, $N_{\mathrm{p}\text{-}90} = 11{,}9$ and $f_{\mathrm{a},0} = 1{,}4$. Lower values may be used for a smaller depth of soil above the pipe, or if the flow only passes over the exposed upper part of the pipe (see References [228] and [371]).

For general impact angles ($0° < \theta < 90°$), the solutions from Formula (A.52) and Formula (A.55) for an engulfed pipe can be used to derive parallel and perpendicular components of force per unit pipe length [357]:

$$\text{Perpendicular force}, F_{\theta,perp} = F_{\theta=90}\sin\theta \qquad \text{and} \qquad \text{Parallel force}, F_{\theta=0}\cos\theta \tag{A.56}$$

Alternatively, a yield envelope approach can be used to evaluate the normal and lateral force coefficients for a given direction of flow, θ. The envelope can be represented by extending the geotechnical approach above to an envelope of the following form [283]:

$$\left(\frac{N_p}{N_{p-90}}\right)^p + \left(\frac{f_a}{f_{a,0}}\right)^q = 1 \text{ with } N_p = N_{p-90}\,(sin\theta)^r \tag{A.57}$$

Using Formula (A.57), values of N_{p} and f_{a} can be found for general values of θ. Separate loading components in the parallel and perpendicular directions are then calculated by substituting these factors in place of $N_{\mathrm{p}\text{-}90}$ and $f_{\mathrm{a}\text{-}0}$ in Formula (A.56), and taking v as the normal component, $v \sin\theta$. The drag coefficient, $C_{\mathrm{D}\text{-}90}$ is replaced for inclined attack to be $C_{\mathrm{D}} = C_{\mathrm{D},90} / \sin\theta$ (see Reference [283]).

Numerical studies provide guidance on the selection of parameters p, q and r for inclined attack (see References [283] and [215]).

A.10.7.2.6 Additional considerations

Formulae (A.51) to (A.57) are for horizontal drag forces in the flow direction. Vertical forces in the form of gravitational and vortex induced vibrations (VIVs) were also observed in the flume experiments and

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numerical analyses. VIV can cause fatigue stresses on pipelines. Reference [362] presents the calculated Strouhal number versus the Re_{nN} from the experiments and numerical analyses.

Glide blocks and out-runner blocks are intact pieces of cohesive sediments that depart from the parent flow often due to hydroplaning and stretching. The framework outlined in A.10.7.9.2 and A.10.7.9.3 can also be applied to determine the flow characteristics of such material for simulation while geotechnical approaches are more suitable for estimating the impact forces. Reference [361] provides an example of the determination of flow characteristics and impact forces.

A.10.7.3 Intervention structures

Intervention structures include anchors, buckling initiators and restraints or supports used to stabilise pipelines and reduce lateral and axial forces.

Intervention structures are designed as subsea foundations complying with the relevant requirements of foundation design as outlined in this document (e.g. shallow, deep, driven or suction pile). Assessment of intervention structures take into account flexibility of the foundation (where applicable), all applied loads and load distributions between multiple foundations (where applicable), site-specific geotechnical conditions and interaction between pipeline and intervention structure.

A.11 Guidance on geotechnical design of anchors for floating structures

A.11.1 General

This clause provides recommendations for the geotechnical design of anchoring systems for floating offshore structures and mobile offshore units. It is applicable to stationkeeping systems with catenary, semi-taut-line or taut-line moorings. Since there is no ISO document available with respect to vertical tethering (TLP structures), reference is made to API RP 2T [1]. The torpedo anchor of Figure A.28 is another example of a vertically tethered anchor with different design recommendations and safety factors than for a TLP.

The options that are available for anchoring floating structures include:

a) drag embedment anchors;

b) anchor piles, comprising driven, suction, vibro-driven, jetted, or drilled and grouted piles;

c) plate anchors, including drag-embedded, or direct-embedded plate anchors;

d) other anchor types, such as gravity anchors and gravity-embedded anchors (free-falling ‘torpedoes’).

The capacity and installation assessment for the anchors should take account of:

— installation methodology and its impact on embedment and mobilised capacity;

— site-specific soil conditions, soil layering, variability in soil conditions and engineering parameters;

— any seabed/subsurface features with potential to adversely affect installation and performance of anchors (e.g. seabed bathymetry, boulders, cemented units);

— Effect of imposed loading regime, e.g. impact of short-term, sustained loading and cyclic/dynamic loading on soil strength. Special care should be taken for soils with potential cyclic degradation such as carbonate or calcareous materials and low plasticity clays;

— Consolidation and drainage characteristics with relevance to installation and loading sequences/conditions;

— Combined loading effects (where applicable) and relevant foundation failure modes;

— Installation tolerance and:

— Scouring effects (where applicable)

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Recommended design criteria and ultimate limit state (ULS) design safety factors for anchoring systems are given in ISO 19901-7. In selecting anchor options, soil conditions, required system performance and reliability, installation and the test loading (where relevant) should be considered. The structural strength of anchors and mooring lines should be demonstrated to be adequate with respect to the required anchoring capacities.

The design of the anchoring system should ensure that allowable limits of stress, displacement and fatigue in the anchor, and cyclic degradation in the surrounding soil are not exceeded during and after installation. The anchoring system above the seafloor should include provisions for inspection and maintenance.

A number of design and installation issues for driven piles, suction piles, plate anchors, and gravity-embedded anchors, all of which are capable of resisting vertical forces, are addressed in A.11.5 through A.11.7. These issues include anchor ultimate holding capacity (UHC) evaluation, installation, and pull testing.

Some of the technological aspects of the design of suction piles, plate anchors and gravity-embedded anchors are still under development. Specific and detailed recommendations are given in this clause to the extent possible. General statements are also used to indicate that some particular aspects should be considered, and references are given for further guidance.

A.11.2 Soil investigation

Seafloor and soil conditions should be investigated for the intended site to provide data for the anchoring system design. Details about equipment and procedures for marine soil investigations are provided in ISO 19901-8.

It is recommended to perform a high quality, high-resolution geophysical survey over the entire areal extent of the foundation system. The survey should use geophysical equipment and practices appropriate to the water depth of interest and provide high-resolution imaging of the seafloor as well as detailed stratigraphic information to a reasonable penetration below the zone of influence of the foundation system. The survey should include the mapping and description of all seafloor and sub-bottom features that can affect the foundation system. This survey should be subjected to a realistic geological interpretation so that it can then serve as a guide to develop a scope of work for the vertical and horizontal extent of the geotechnical investigation (i.e. number, depth, and location of soil borings and/or in situ tests, such as cone penetrometer tests and CPTU) and to aid in the interpretation of the acquired geotechnical data. Further information on geophysical marine survey is provided in ISO 19901-10.

The stratigraphic data thus obtained should be integrated with geotechnical data collected subsequently, or with existing geotechnical data (if any), to assess constraints imposed on the design by geological features, and to allow for soil data interpolation and/or extrapolation in the event the anchor locations are shifted due to changes in mooring line lengths and/or headings, field layout, platform properties, and mooring leg properties. Guidance about the integration of geophysical and geotechnical data is provided in ISO 19901-10.

The sampling and in situ testing scope and intervals should ensure that each significant stratigraphic layer is characterized. The minimum vertical extent of the site investigation should be related to the expected zone of influence of the actions imposed by the base of the foundation and should exceed the anticipated design penetration by at least the anchor diameter or anchor fluke width. If reverse end bearing (REB) at the suction anchor pile tip is taken into account in the vertical capacity analysis, soil characterization up to three diameters for suction piles or three fluke widths for plate anchors below the design penetration depth is more appropriate. It is critical to ensure that no high-permeability layers are present within the zone influenced by the mobilization of REB, particularly if the anchor is expected to resist long-duration forces, such as those imposed by loop currents.

A.11.3 Anchor types

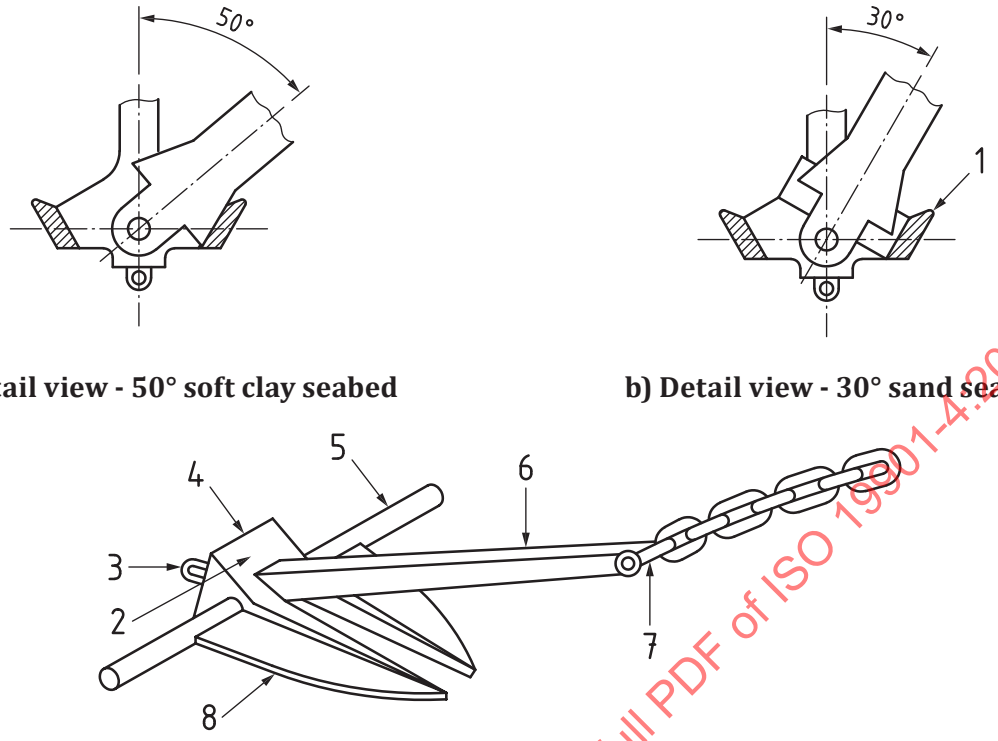
A.11.3.1 Drag embedment anchors

Traditional drag embedment anchors (Figure A.23) were initially used for mobile (temporary) mooring operations. Drag anchor technology has advanced considerably. Engineering and testing indicate that the new generation of fixed fluke drag anchors develops high holding power even in soft soil conditions. A high efficiency drag anchor is generally considered to be an attractive option for mooring applications because of

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its easy installation and proven performance. The anchor section of a mooring line can be preinstalled and test loaded prior to floating structure installation.



a) Detail view - 50° soft clay seabed

b) Detail view - 30° sand seabed

c) General view

Key

1	adapter block	4	crown	7	shackle
2	tripping palm	5	stock	8	fluke
3	crown padeye	6	shank		

Figure A.23 — Traditional drag embedment anchor

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A.11.3.2 Driven anchor piles

The resistance of a driven anchor pile to uplift and lateral loading is primarily a function of pile dimensions, the manner in which the pile is installed and loaded, and the type, stiffness, and strength of the soil adjacent to the pile. Horizontal capacity can be increased considerably by adding special elements, such as skirts or wings to the pile top. Driven anchor piles can be designed to develop high lateral and vertical resistances, and be very stable over time.

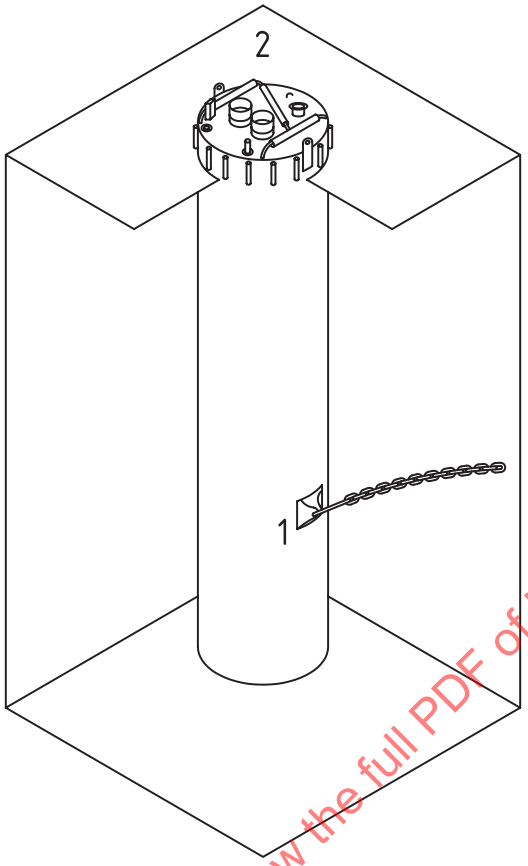
Vibro-driving (see Reference [187]), jetting (see Reference [179]) or drilling and grouting techniques can be considered for other types of anchor piles. Disturbance of soil during vibro-driving, jetting or drilling operations should be evaluated.

A.11.3.3 Suction anchor piles

Suction anchor piles can be used for large deep-water mooring systems and can be designed for very high mooring line tensions. They are typically tall steel cylindrical structures with or without internal stiffener systems. The cylinder unit is open at the bottom and closed at the top (see Figure A.24). A suction anchor pile is installed by first lowering it into the soil to self-penetration depth (i.e. penetration due to the submerged pile weight). The remainder of the required penetration is achieved by pumping the trapped water from the inside of the pile. The differential pressure thus created (generally called ‘under-pressure’ or ‘suction’) results in an additional driving force on the anchor top, which drives the pile into the soil. As the penetration

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increases, the driving force needed normally increases, requiring a gradually increasing differential pressure.



Key

1 padeye

2 anchor top cover with installation aids, venting hatches and anodes

Figure A.24 — Suction anchor pile

After reaching design penetration, the water outlet is closed, allowing the suction anchor pile to achieve substantial capacity to resist horizontal forces, vertical uplift forces, moments and combinations of these.

For suction anchor piles embedded in clay and with a closed outlet, the capacity to resist mooring line tensions is governed by the undrained shear strength of the soil around and beneath the anchor. The capacity depends on depth of penetration, anchor diameter, shear strength of the clay, shear strength at the clay–anchor wall interface, mooring line inclination, and the location of the attachment point. In the case where the top part is left open or retrieved, or for long-term uplift components, pull-out of the anchor can also be a possible failure mechanism.

The holding capacity is generally greater if the anchor pile is prevented from tilting. To avoid tilting, the line attachment point can be lowered from the top of the anchor to a point on the anchor wall at an optimal depth below the seafloor. The location of the optimal line attachment point depends on the shear strength profile, the shear strength at the clay–anchor wall interface, the mooring line inclination, the submerged anchor weight, and the depth-to-diameter ratio of the anchor. The optimal location is typically two-thirds to three-quarters of the length of the anchor pile downwards from the seafloor.

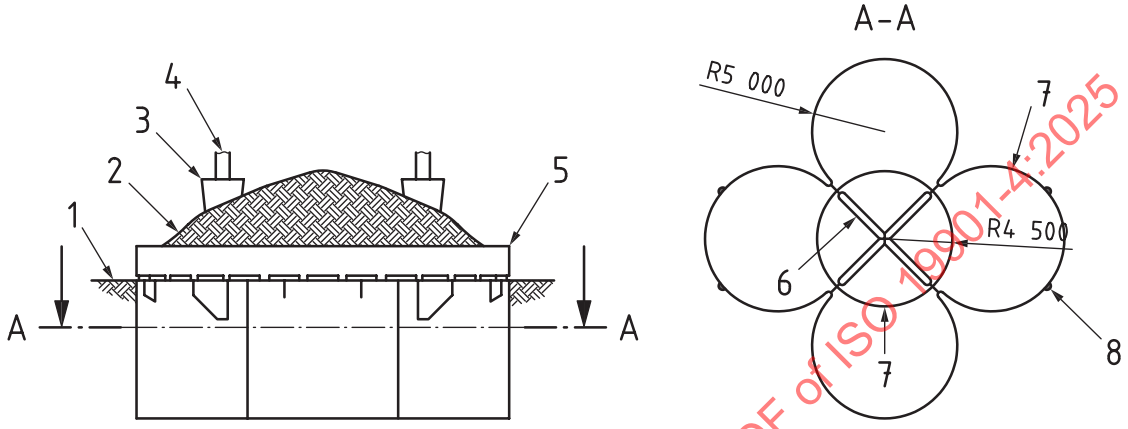
As suction anchors are shallow structures compared with driven piles, deep soil borings are not required, but more detailed soil data are needed at shallow depths than for driven piles. Suction anchors have mainly been applied in cohesive clay type soils. Suction embedment penetration through sand or granular layers is feasible, provided the suction anchor design takes this into account [326]. Penetration into non-cohesive granular type soils requires special considerations which are described in A.11.5.2.2.1.

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Suction anchor length-to-diameter ratios can vary from 2:1 for stiff clay soils to as much as 7:1 for very soft clay soils. Suction anchors are often designed with large depth-to-diameter ratios in soft clays since the upper part of soft clay deposits provides limited bearing capacity and skin friction.

A suction caisson is a suction anchor that is relatively shallow in height and is designed for relatively small penetration. The submerged weight of the suction caisson can make up a large part of the anchor's vertical holding capacity. A multi-cell concrete structure with a large footprint and a shallow skirt penetration is an example of suction caisson (see Figure A.25 and Reference [318]). The vertical capacity is derived mainly from its self-weight plus possibly some skin friction and internal suction. Horizontal resistance is generated by skirt penetration and friction between the soil layers subject to shear.



Key

1	seafloor	5	parapet wall
2	iron ore	6	stiffeners (beam)
3	tendon receptacle	7	anchor skirt
4	tendon	8	towing padeye

Figure A.25 — Suction caisson anchor

A.11.3.4 Plate anchors

A.11.3.4.1 General

Plate anchors were initially used by the US Navy for the anchoring of fleet mooring buoys. They are installed at deep penetration beneath the seafloor where the generally higher soil strength allows the use of relatively small plate anchors for high mooring actions. Plate anchors typically have significant vertical holding capacity. This allows the use of taut-leg mooring systems where the anchor line can intersect the seafloor at significant inclinations. Plate anchors can be placed in two broad categories: drag-embedded and direct-embedded.

A.11.3.4.2 Drag-embedded plate anchors

Drag-embedded plate anchors are embedded to deep penetration in a manner similar to drag anchors. During installation, the anchor is first placed on the seafloor, and as the anchor is pulled along the seafloor, it penetrates the soil. Initially, the anchor dives more or less parallel to the fluke, progressively rotating until the target depth is achieved. Following embedment, the anchor fluke is oriented such that it becomes nearly perpendicular to the mooring line and applied loading (a process called 'keying' or 'triggering'), providing high horizontal and/or vertical holding capacity depending on the orientation of the line.

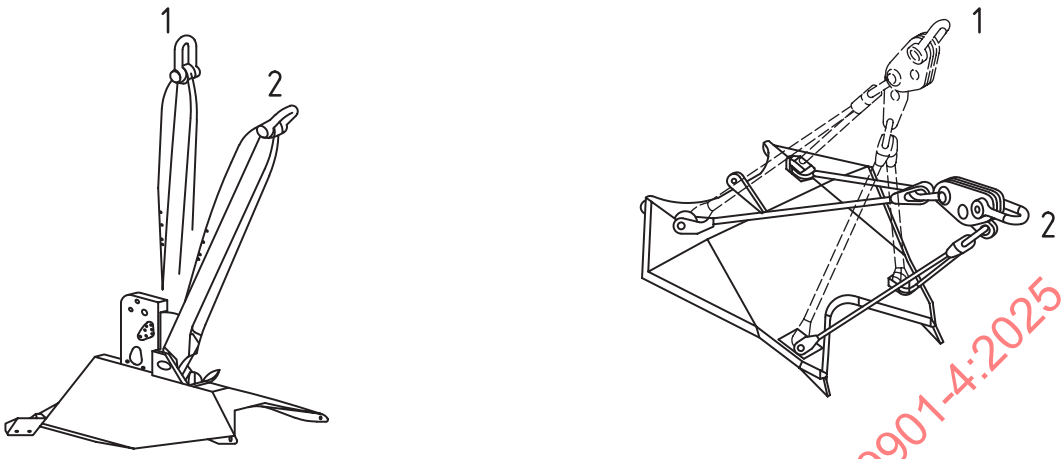
These drag-embedded plate anchors are often referred to as vertically loaded anchors (VLA). Two VLAs are commonly used by the offshore industry: Stevmanta®[1)] (see Reference [295]) and DENNLA®[1)] (drag-

1) These anchors are examples of suitable products available commercially. This information is given for the convenience of users of this document and does not constitute an endorsement by ISO of these products.

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embedded near normally loaded anchor) (see Reference [130]). The Stevmanta® anchor uses a bridle system to convert from its installation configuration to its plate anchor operational orientation whereas the DENNLA®[1)] anchor uses an articulated shank (Figure A.26).



a) Dennla Mk4® **b) Stevmanta®**

Key

1 normal (or near normal) loading mode

2 installation mode

Figure A.26 — Drag-embedded plate anchors (VLA)

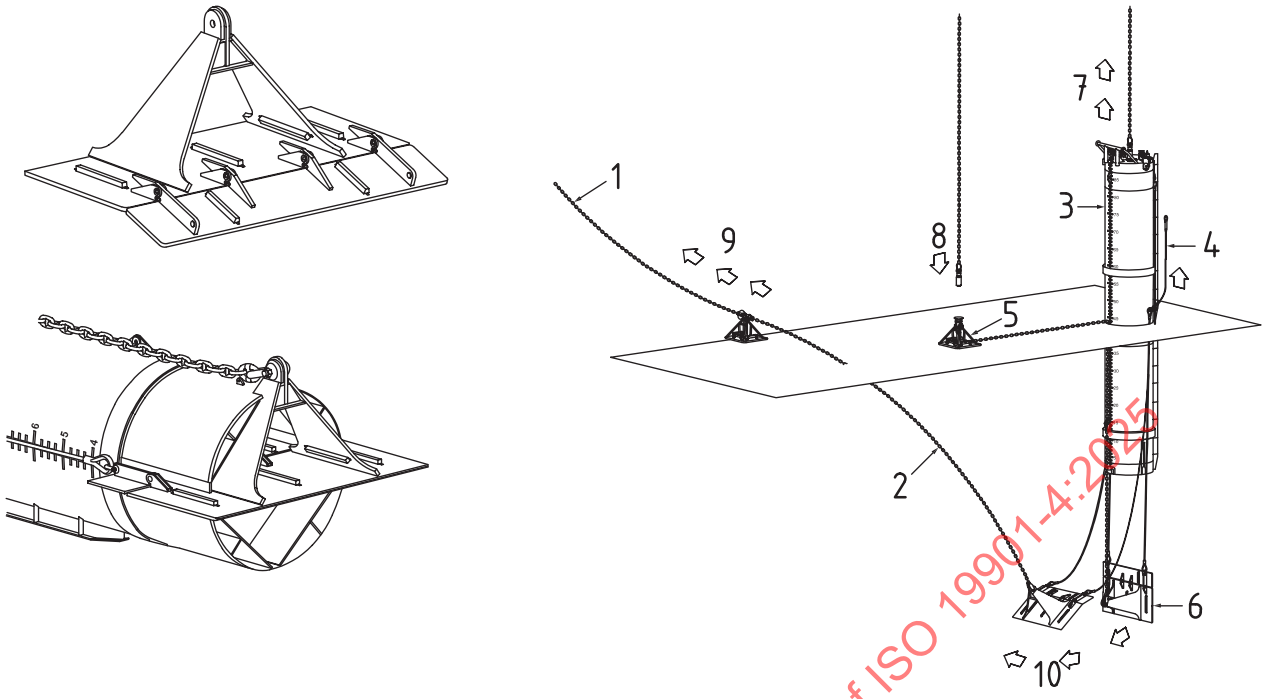
A.11.3.4.3 Direct-embedded plate anchors

Direct embedment of plate anchors can be achieved by suction, impact or vibratory hammer, propellant, or hydraulic ram.

Suction-embedded plate anchors have been used for major offshore mooring operations. As an example, the suction-embedded plate anchor (SEPLA®[1)]) uses a so-called 'suction follower' which is essentially a reusable suction anchor with its tip slotted for insertion of a plate anchor. The suction follower is retracted by reversing the pumping action once the plate anchor achieves its design depth, and can be used to install additional plate anchors (Figure A.27). In the SEPLA® concept, the fluke of the plate anchor is embedded in vertical position, and adequate fluke rotation is achieved during a keying process by pulling on the mooring line [348].

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a) Plate anchor **b) Installation**

Key

1	mooring line	6	SEPLA® anchor
2	forerunner chain	7	recovery of suction follower
3	suction follower	8	docking subsea connector
4	retainer/recovery lines	9	tensioning mooring line
5	subsea connector mudmat	10	keying plate anchor

NOTE This anchor is an example of suitable product available commercially. This information is given for the convenience of users of this document and does not constitute an endorsement by ISO of this product.

Figure A.27 — Suction-embedded plate anchor (SEPLA®)

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A.11.3.5 Gravity anchors

Gravity anchors are deadweight anchors which commonly consist of concrete or steel blocks, scrap metal or other materials of high density. Their uplift capacity is primarily derived from their own submerged weight. Their horizontal capacity is a function of the friction between the anchor and the soil and of the shear strength of the soil beneath the anchor. Gravity anchors are generally used in relatively hard seabed conditions where other types of anchors cannot penetrate. Gravity anchors are typically more suitable for small mooring systems in relatively shallow water, where holding capacity requirements are limited and/or where ballast can be added relatively easily to the gravity anchor. Gravity anchors are generally not used for large deep-water mooring systems.

Gravity anchors can be designed with skirts for enhanced horizontal capacity, to improve the bearing capacity (if needed), to mitigate piping below the edges of the structure base induced by rocking motions and to ensure that scour does not undermine the base.

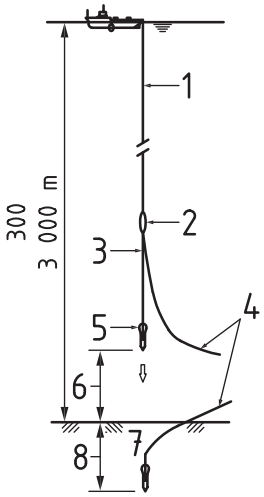
A.11.3.6 Gravity-embedded (free-falling) anchors

Gravity-embedded anchors are commonly shaped as ‘torpedo’ steel structures which penetrate the seabed by free-falling and are used as anchoring solution in soft clayey soils. The anchors are lowered by means of an installation line to a designated free-fall drop height above seafloor and penetrate to the target depth

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below seafloor by kinetic energy obtained during the free-fall (see Figure A.28 and References [46], [213] and [368]).

Gravity-embedded anchors obtain significant horizontal and inclined holding capacity by lateral soil resistance against the wings and friction along the soil–steel interface.



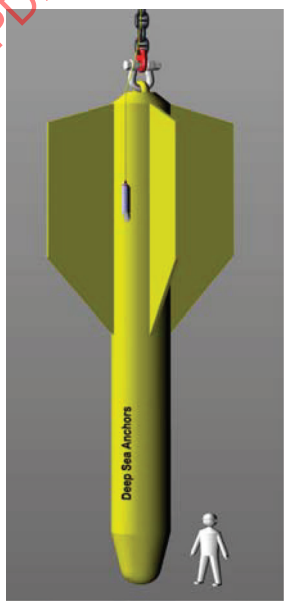
a) Installation principle



b) 'Torpedo' pile [46]



c) OMNI-Max anchor [372]



d) Deep penetrating anchor [153]

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Key

1	installation line	4	permanent mooring line	7	seafloor
2	release unit	5	anchor	8	penetration depth
3	lead line chain	6	drop height		

NOTE These anchors are examples of suitable products available commercially. This information is given for the convenience of users of this document and does not constitute an endorsement by ISO of these products.

Figure A.28 — Gravity-embedded free-falling anchors

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A.11.4 Geotechnical design of drag anchors

A.11.4.1 General

Recommended safety factors for holding capacity of drag anchors are given in ISO 19901-7:2013, Table 6.

When used for mobile moorings, the design safety factors for drag anchors are substantially lower than those for mooring line tensions. The rationale is to allow the anchor to move instead of the mooring line breaking in the event of mooring over-loading. Anchor movements of the most heavily loaded lines would normally cause favourable redistribution of the mooring line tensions. This is expected to help the mooring system survive environmental actions exceeding those from the ULS design situation.

Evaluation of the holding capacity of drag anchors is addressed in this subclause and in Reference [105].

The holding capacity of a drag anchor in a particular soil condition represents the maximum horizontal steady pull that can be resisted by the anchor at continuous drag. This includes the resistance of the chain or wire line into the soil for an embedded anchor but excludes the friction of the chain or wire on the seafloor.

Drag anchor holding capacity is a function of several factors, including:

a) Anchor type: fluke area, fluke angle, fluke shape, anchor weight, tripping palms, stabilizer bars, etc. Figure A.29 shows drag anchors commonly used by the offshore industry.

b) Anchor behaviour during deployment: opening of the flukes, penetration of the flukes, depth of burial of the anchor, stability of the anchor during dragging, soil behaviour over the flukes, etc.

c) Seabed conditions. Note the specific guidance given for carbonate soils (see 6.3.3 and A.11.4.11).

Furthermore, a long drag distance can be required for an anchor to reach full penetration and develop the ultimate holding capacity. This can be acceptable for anchoring a drill rig in an open water location, but is likely to be unacceptable for a production location where the seafloor is congested with subsea installations.

Due to the wide variation of these factors, predicting the holding capacity of a drag anchor is difficult. A verification of the holding capacity can be determined after the anchor is deployed and test loaded. If drag anchor holding capacity does not meet design capacity, contingency measures should be taken into account.

Anchor performance data for the specific anchor type and soil condition should be obtained if possible. In the absence of credible anchor performance data, Figure A.30 and Figure A.31 can be used to estimate the holding power of drag anchors commonly used to moor floating vessels, acknowledging that the holding capacity curves in Figure A.36 and Figure A.37 do not include a design safety factor and are based on the assumption that the anchor reaches full penetration.

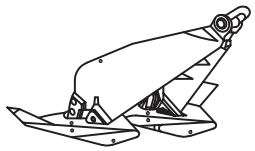
Figure A.31 and Figure A.32 are reproduced from Reference [22], except that the holding capacity curves for the Moorfast (or Offdrill II) and Stevpris®[1] anchors were upgraded, based on model and field test data and field experience. The design curves presented in Figure A.30 and Figure A.31 represent, in general, the lower bounds of the test data. The tests used to develop the curves were performed at a limited number of sites. As a result, the curves are for use in generic soil types such as soft clay (i.e. normally consolidated clay with undrained shear strength increasing monotonically with depth) and sand and should not be applied to carbonate soils.

Recent studies indicate, however, that several parameters, such as soil strength profile, lead-line type (wire rope versus chain), cyclic actions and anchor soaking, can significantly influence anchor performance in soft clay. Also, some high efficiency anchors have demonstrated substantial resistance to vertical actions in soft clay. Furthermore, there are new versions of high efficiency anchors that are not covered by Figure A.30 and Figure A.31.

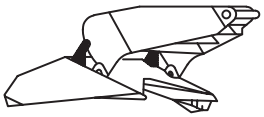
As Figure A.30 and Figure A.31 only provide anchor holding capacity estimates, more detailed analyses are needed, if uncontrolled anchor drag cannot be tolerated in congested subsea locations where it can cause damage to existing subsea installations. If it is impractical to apply an installation tension required to completely avoid future anchor drag, it can be necessary to demonstrate that the extent of anchor drag that can occur will not encroach on the existing subsea installations in the area.

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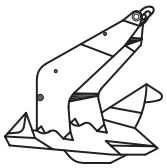
a) Stevpris® Mk. 5



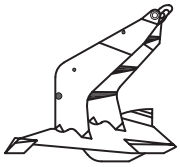
b) Stevpris® Mk. 6



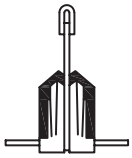
c) Stevshark® Mk. 5



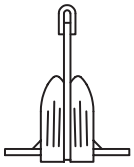
d) Bruce FFTS® Mk.4



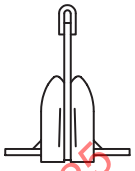
e) Bruce FFTS® PM



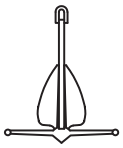
f) Navmoor



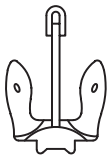
g) Stato



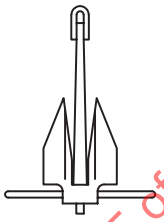
h) Moorfast Offdrill II



i) LWT



j) Stockless



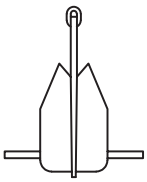
k) Danforth/GS (type 2)



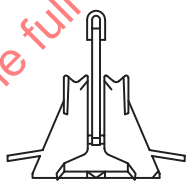
l) Bruce, TS



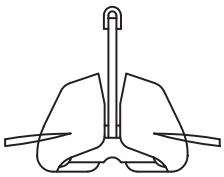
m) Bruce, cast



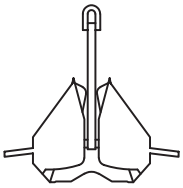
n) Boss



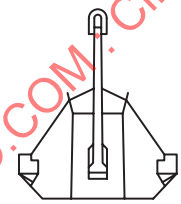
o) Stevdig



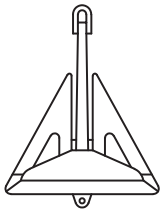
p) Stevmud



q) Stevfix



r) Hook



s) Flipper delta

NOTE These anchors are examples of suitable products available commercially. This information is given for the convenience of users of this document and does not constitute an endorsement by ISO of these products.

Figure A.29 — Drag embedment anchors

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A.11.4.2 Effect of shear strength gradient in clay

Centrifuge test data, as well as results from analytical studies using a calibrated drag embedment anchor prediction tool, indicate that a more or less linear relationship exists between the anchor holding capacity and the shear strength gradient of the clay [114]. Significant deviations from this linear relationship are observed when the shear strength seafloor intercept and/or the sensitivity of the clay are varied in addition to the shear strength gradient. In general, the effect of the various parameters on the anchor holding capacity in clay accentuates with increasing degree of mobilisation of the anchor capacity. This relationship also varies with the anchor type and anchor size.

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Due to the complexity of the problem, a reliable, calibrated prediction tool that can take all influencing parameters into account should be used to establish a basis for design of drag embedment anchors [91].

A.11.4.3 Effect of lead-line type in clay

Field tests and analytical studies indicate that in soft clay, when the lead-line is steel wire line, an anchor can penetrate deeper and give significantly higher holding capacity than when a chain lead-line is used. For the limited cases studied, an anchor connected to steel wire line provided 15 % to 40 % higher holding capacity than the same anchor connected to chain. This is in good agreement with the results from a full-scale test programme. The studies were limited to high efficiency anchors in soft clay with a fairly constant shear strength gradient. A side effect is that the required anchor installation tension is reached with less drag, if a wire lead-line is used instead of a chain lead-line.

A.11.4.4 Effect of cyclic loading in clay

Cyclic loading affects the static undrained shear strength, s_u, in two ways.

a) During a storm, the rise time from mean to peak loading can be about 3 s to 5 s (1/4th of a wave frequency tension cycle), as compared to 0,5 h to 2 h in a static consolidated undrained triaxial test. This higher loading rate leads to an increase in the undrained shear strength and, consequently, in the anchor holding capacity.

b) As a result of repeated cyclic loading during a storm, the undrained shear strength decreases. The degradation effect increases with increasing over-consolidation ratio of the clay.

The cyclic shear strength values used in geotechnical design are generally based on cyclic laboratory tests with periods of typically 1 s to 10 s and therefore account for both these effects.

More information about the prediction of cyclic loading effects are provided in References [105], [106], [15] and [1]. More general considerations about the effects of cyclic loading in clay are described in A.8.3.2.3.

A.11.4.5 Effect of anchor soaking in clay

Soil set-up due to thixotropy can lead to a significant increase in anchor holding capacity in a few hours or days after installation (see for example results from temporary stoppage during instrumented field tests reported in Reference [93]). Over the subsequent weeks, soil set-up due to thixotropy effects gradually increases in combination with soil consolidation (dissipation of excess pore water pressure).

Generally speaking, drag embedment anchors should therefore be installed without stoppage. A temporary interruption before reaching the prescribed installation tension can prevent further anchor penetration, if the increased tension required to restart the anchor after stoppage is higher than the pull available from the installation equipment. The consequence is that the long-term anchor capacity is no higher than that given by the installation tension of the initial step plus the increase due to post-installation effects (thixotropy/consolidation and cyclic loading effects). On the other hand, once the anchor starts to drag after a set-up period, this effect disappears completely.

In a design situation, in which the anchor installation tension is intended to ensure stationkeeping of a floating structure without anchor drag, a safety factor should be applied to the predicted post-installation effects (set-up and cyclic loading), and an adequate overall safety margin should be considered to determine the installation tension meeting such design requirements. In this case, the set-up effect can represent a significant contribution to the total holding capacity, which should be reduced for anchor penetration depths less than 2,5 fluke widths and should be set to zero, if the fluke penetration depth is very shallow (see further discussion in Reference [105]).

A.11.4.6 Capacity in clay under inclined line loading

For deeply embedded drag embedment anchors (>2 to 2,5 fluke widths) the allowable uplift angle at the seafloor for ULS intact condition or redundancy checks can be as high as 20°, if proper anchor installation analyses have shown that the uplift angle at the seafloor is significantly less than the uplift angle at the anchor padeye.

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It is not advisable to apply a high uplift angle at the seafloor during the initial shallow penetration of the anchor; otherwise, full penetration depth of the anchor can possibly not be achieved. After reaching a penetration depth greater than 2 to 2,5 fluke widths, the mooring line uplift angle at the seafloor can be gradually increased. This issue is discussed in some detail in Reference [105].

Significant evidence supports the use of a non-zero uplift angle at the seafloor on drag embedment anchors that penetrate sufficiently deep into soft clay. The following additional guidelines are proposed in this respect.

a) Uplift angles at the seafloor should not be accepted for certain operations with mobile moorings where the soil conditions have not been thoroughly investigated or the anchor installation tension is insufficient to ensure deep anchor penetration.

b) The maximum uplift angle at the seafloor should be assessed in accordance with the principles outlined herein under the design situations for the ULS intact and redundancy checks.

c) A zero uplift angle should be maintained until the recommended minimum anchor penetration depth has been reached.

d) The anchor holding capacity should be reduced by a factor R, which is a function of the seafloor uplift angle, and accounts for the reduced friction due to shorter embedded line length. The R values in Table A.10 are applicable for Bruce FFTS® Mk. IV and Stevpris® Mk. V anchors.

Table A.10 — R values for Bruce FFTS® Mk. IV and Stevpris® Mk. V anchors

Seafloor angle (°)	0	5	10	15	20
R	1,0	0,98	0,95	0,89	0,81

In areas that are susceptible to seabed trenching, the effect of possible trenching in combination with uplift should be evaluated to ensure sufficient anchor capacity.

For taut-leg mooring systems, where mooring lines with seafloor angle greater than 20° impose significant vertical forces on the anchor at all times, a typical solution is to use anchor piles or plate anchors for which design guidelines are provided in A.11.5 and A.11.6.

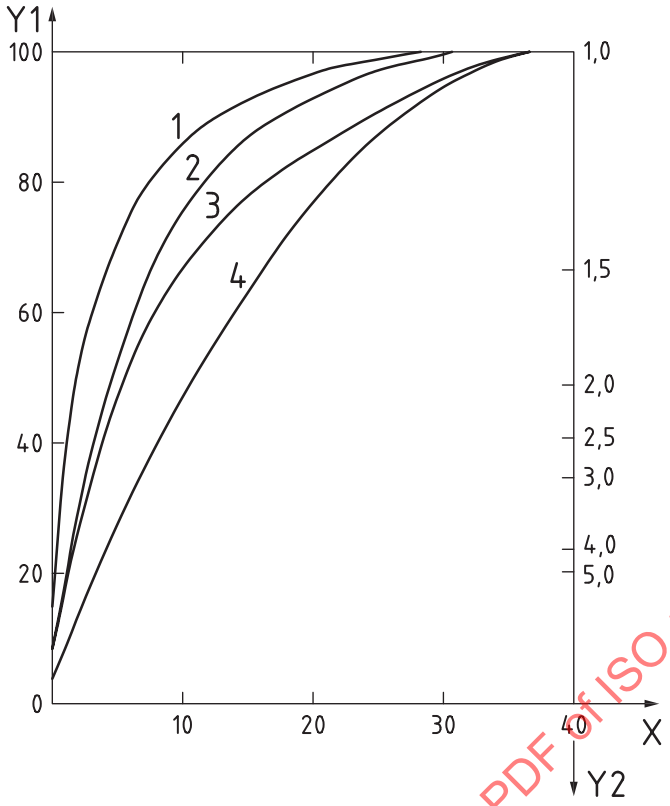
A.11.4.7 Drag distance and penetration depth in soft clay

Many factors affect the drag–penetration depth relationship, including site-specific soil data (e.g. soil stratigraphy, seafloor shear strength, average shear strength gradient, soil sensitivity), and the size and type of anchor. For screening-level analysis, drag distance and penetration depth estimates from Reference [249] are presented in Figure A.30 and Table A.11, respectively. This information is valid for chain lead-lines and shear strength gradients of 1,4 kPa/m to 2,0 kPa/m. Deviation from this range can affect these values, especially the penetration depth estimates.

If the anchor design relies on further penetration to reach holding capacity, the additional drag to resist the design intact actions should not over-load neighbouring lines.

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Key

X	drag distance/ fluke length	2	hook anchor
Y1	percent of maximum capacity	3	anchor types Bruce FFTS® Mk. III / Bruce TS / Danforth / GS (type 2)[a] / LWT[a] / Moorfast / Navmoor / Offdrill II[a] / Stato / Stevmud / Stevpris® Mk. III
Y2	corresponding safety factor	4	anchor types Boss[a] / Flipper Delta[a] / Stevdig[a] / Stevin[a]
1	stockless anchor (fixed)	[a]	Assumed based on geometric similarities.

Figure A.30 — Holding capacity versus drag distance in soft clays [249]

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Table A.11 — Estimated maximum fluke tip penetration [249]

Anchor type	Normalized fluke tip penetration (fluke lengths)	
	Sands/Stiff clays	Mud (e.g. soft silts and clays)
Stockless (fixed fluke)	1	3
Moorfast Offdrill II	1	4
Boss Danforth Flipper Delta GS (type 2) LWT Stato Stevfix	1	4,5
Stevpris® Mk. III Bruce FFTS® Mk. III Bruce TS Hook Stevmud	1	5

A.11.4.8 New anchor designs

New anchor designs and improvements to existing anchors continue to be developed. However, well controlled instrumented tests and field performance data are insufficient for predicting the performance of many of these innovative high efficiency anchors, although results from such tests can still be used to calibrate anchor prediction tools (see A.11.4.9). Just as important as the ultimate holding capacity is the ability to predict drag–penetration–tension relationships for mobilised loadings, which are much less than the ultimate holding capacity. In the absence of better information, the holding capacities of these new anchors can be conservatively estimated from:

$$H_n = H_s(A_n/A_s)^n \quad \text{(A.58)}$$

where

H_n is the holding capacity of new design;

H_s is the holding capacity of reference design (e.g. Bruce FFTS® Mk. III or Stevpris® Mk. III in Figure A.31 and Figure A.32) of the same weight;

A_n is the fluke area of new design;

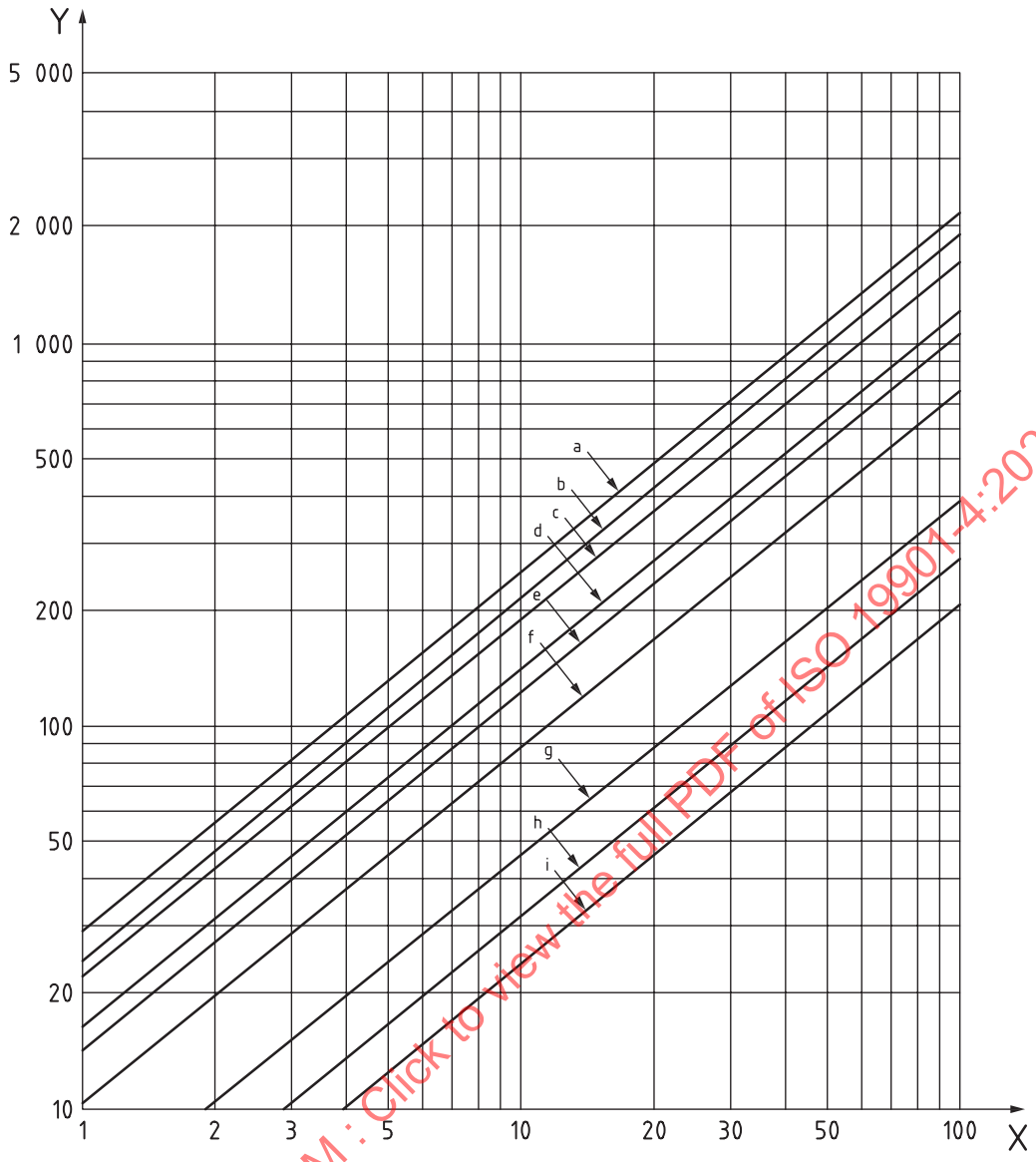
A_s is the fluke area of reference design of same weight;

n is the 1,4 factor commonly used for high efficiency drag anchors.

The fluke area ratio A_n/A_s can be obtained from anchor manufacturers.

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Key

X anchor weight (kips)

Y anchor holding capacity (kips)

a Bruce FFTS Mk. III, Stevpris Mk. III.

b Navmoor, Stato, Boss.

c Bruce TS, Hook, Stevfix.

d Flipper Delta, Stevin, Stevdig.

e Moorfast, Offdrill II.

f Danforth, GS, LWT.

g Stockless, fixed fluke.

h Bruce, cast.

i Stockless, movable fluke.

NOTE 1 Fluke angles set for soft clay seafloor condition as per manufacturer's specification

NOTE 2 1 kip = 4,448 kN.

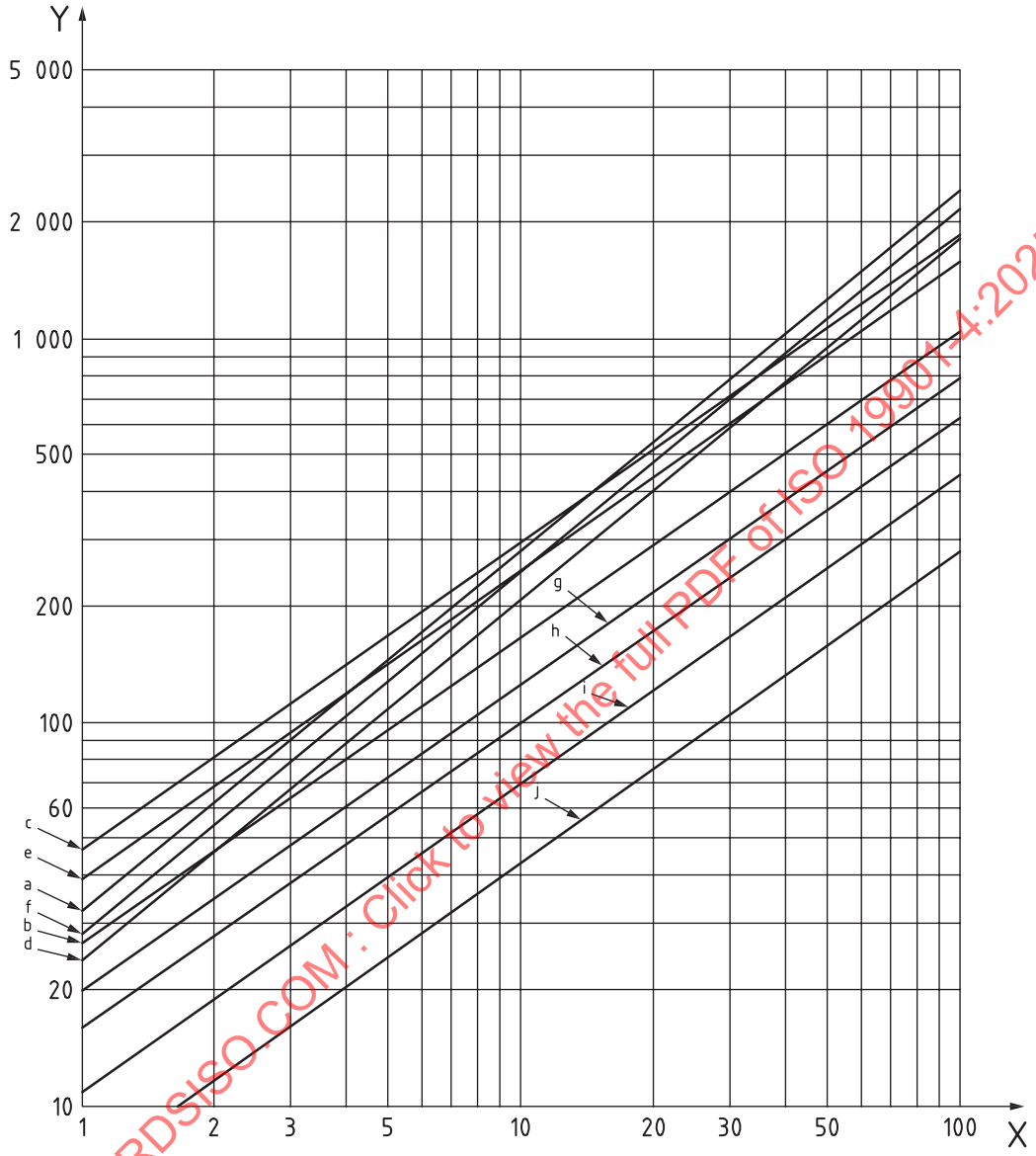
NOTE 3 This figure was reproduced from Reference [249], except that the holding capacity curves for the Moorfast (or Offdrill II) and the Stevpris anchors were upgraded. The design curves reflect data valid for anchor designs as of 1987. New anchor designs have since been developed, but the curves for these new designs were not included. The design curves in this figure do not include a design safety factor and assume that the anchor reaches design penetration (see A.11.4.1).

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NOTE 4 These anchors are examples of suitable products available commercially. This information is given for the convenience of users of this document and does not constitute an endorsement by ISO of these products.

Figure A.31 — Anchor system holding capacity in soft clay



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Key

X anchor weight (kips)

Y anchor holding capacity (kips)

Fluke angles set for sand seafloor condition as per manufacturer's specification.

a Navmoor, Boss.

b Stevin.

c Stevfix, Stevdig.

d Stevpris, straight shank, Bruce TS.

e Bruce, cast.

f Stato, 30° pulse angle.

g Danforth GS, LWT.

h Moorfast, Offdrill II, 20° fluke angle, Hook.

i Stockless, 35° fluke angle.

j Stockless, 48° fluke angle.

NOTE 1 1 kip = 4,448 kN

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NOTE 2 This figure was reproduced from Reference [249]. The design curves reflect data valid for anchor designs as of 1987. New anchor designs have since been developed, but the design curves for these new designs were not included. The design curves in this figure do not include a design safety factor and assume that the anchor reaches design penetration (see A.11.4.1).

NOTE 3 These anchors are examples of suitable products available commercially. This information is given for the convenience of users of this document and does not constitute an endorsement by ISO of these products.

Figure A.32 — Anchor system holding capacity in sand

A.11.4.9 Analytical tools for anchor performance evaluation

Analytical tools based on limit equilibrium principles for anchor embedment and capacity calculation in soft clay are available. These tools allow modelling of different anchor designs and provide detailed anchor performance information, such as anchor movement trajectory, anchor rotation, mooring line profile below the seafloor and ultimate anchor capacity. Recommendations for these tools to yield reliable predictions include the following.

— The analytical tool should be calibrated against results from high quality instrumented tests by field testing or centrifuge testing performed on the type of anchor of interest.

— The soil properties should be well known, which is not necessarily the case when designing and installing drag embedment anchors. Where the soil properties are uncertain, suitable upper and lower bound soil parameters should be established, and the anchor design should be based on the more conservative prediction.

— Users should be aware of the tool's limitations and should be familiar with mooring operations. For example, some tools typically show that the anchor penetration increases continuously, leading to higher and higher anchor holding capacity. In such cases, the user should consider limiting the drag distance for calculating the anchor holding capacity to a distance that does not result in unacceptable vessel excursions.

— Empirical formulae or field experience, if available, should be used to support analytical predictions.

— The analytical tool should be able to handle layered soil profiles. Some tools can handle layered clay profiles with sand layers of limited thickness while other tools cannot model layered soil profiles.

A.11.4.10 Anchor holding capacity in sand

No significant study on the behaviour of drag embedment anchors in sand has been carried out since the US Navy's study [249]. Anchors do not achieve deep penetration in sand and no uplift resistance can be relied upon from shallow penetration anchors in any soil conditions, i.e. the line uplift angle at the seafloor should be zero. Moreover, scour effects on anchor embedment should be taken into account when drag anchors installed in sand are still visible at the seafloor.

Contrary to anchors in soft clay, anchors in sand do not gain any additional capacity from post-installation effects due to thixotropy, consolidation or cyclic loading effects. In this case, the initial anchor installation tension should be set high enough to provide the required safety factor for the anchors and the mooring system accounting for the uncertainty in the loading calculation [105].

The limited capacity of mobile offshore drilling units (MODU) winches can result in shallow anchor penetration during installation in dense sand. In some cases anchors are visible at the seafloor after installation. In the cases of shallow penetration, it is not recommended to assume that the anchors continue to penetrate upon overloading. Anchor penetration during installation should be estimated and the anchor capacity at estimated penetration depth should be evaluated using the analytical method presented in Reference [251].

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A.11.4.11 Anchor holding capacity in soils other than soft clay and sand

Predicting anchor holding capacity in hard clay, calcareous sand, coral or rock seafloor and layered soil profiles is complex and is dependent upon the detailed soil/rock data for the location of each anchor cluster. In these soils/rocks, the anchor penetration is often very shallow, which means that the same precautions as recommended for anchors in sand (see A.11.4.10) should be followed.

Figure A.30 and Figure A.31 should not be used for carbonate and calcareous materials. Anchor performance in similar seabed conditions or limit equilibrium methods should be used for design as described in Reference [71]. In carbonate or calcareous materials, the effect of the imposed loading regime (e.g. impact of short-term, sustained loading and cyclic/dynamic loading on soil strength) should be taken into account (potential cyclic degradation) as described in A.6.3.

A.11.4.12 Holding capacity generated by friction along the mooring line

The holding capacity generated by friction of chain and steel wire line on the seafloor can be estimated by:

$$P_{cw} = f L_{cw} W'_{cw} \tag{A.59}$$

where

P_{cw} is the chain or wire line holding capacity;

f is the coefficient of friction between chain or wire line and the seafloor;

L_{cw} is the length of chain or wire line in contact with the seafloor;

W'_{cw} is the submerged unit weight of chain or wire line.

The coefficient of friction depends upon the nature of the seafloor and on the type of mooring line. Static (starting) friction coefficients are normally used to compute the holding power of the line and sliding coefficients are normally used to compute the friction forces on the line during mooring deployment.

If more specific data are not available for chain and wire line, the generalized coefficients given in Table A.12 can be used for various seafloor conditions, such as soft mud, sand and clay. Guidance for calculation of the seabed friction is also provided in Reference [105]. However, industry experience indicates that coefficients of friction can vary significantly for different soil conditions, and higher values for the sliding coefficient of friction have been encountered. In calcareous and carbonate materials, lower values are possible (see Reference [1]), coupled with low chain embedment in dense or cemented materials.

NOTE Considerations about capacity generated by friction along the mooring line apply to all types of anchors with deeply embedded anchor line attachment padeye, not to drag anchors only.

Table A.12 — Mooring line friction coefficients

Anchor type	Coefficient of friction f	
	Static	Sliding
Chain	1,0	0,7
Wire line	0,6	0,25

A.11.4.13 Installation of drag anchors

Drag anchor installation tolerances should be established and should be considered in the anchor's geotechnical, structural, and installation design. Typical tolerances to be considered are:

— allowable deviation from target heading of the mooring line attachment to limit padeye side loadings and rotational moments on the anchor padeye;

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— minimum penetration required before test loading to achieve the required holding capacity.

For drag anchors used in permanent moorings, the anchor design should incorporate adequate installation information to ensure that the anchor has reached the target penetration depth, thereby meeting the safety requirements of the mooring system for the actual soil and design situations. Typical information to be monitored and recorded includes:

a) drag anchor installation line tension versus time;

b) catenary shape of installation line based on line tension and line length to verify that uplift at the seafloor during embedment is within allowable ranges and to verify anchor position;

c) direction of anchor embedment;

d) drag distance;

e) final anchor penetration depth.

Acceptance criteria for drag anchors used in mobile (i.e. temporary) moorings should be established on a case-by-case basis.

A.11.4.14 Out-of-plane loading of anchors

In areas prone to revolving storms, MODUs experiencing a one or multiple line failure can exert forces on the anchor that are out-of-plane to the installation direction. This can cause a reduction in the holding capacity of the drag anchor and potentially anchor dragging [140]. On the other hand, drag anchors are also capable of generating significant holding capacity in the out-of-plane direction.

For MODU moorings where the possible consequences of out-of-plane loading are seen as a risk, possibly due to nearby infrastructure, this should be taken into account in the design of the anchoring system. Selection of the appropriate drag anchor in combination with sufficient installation tension to fully embed the anchor, can increase the out-of-plane resistance of the drag anchor. Out-of-plane loading can also cause additional drag distance to reach ultimate holding capacity and should be considered in the design of the anchor pattern. The structural capacity of the drag anchor should also be evaluated for out-of-plane loading.

A.11.5 Geotechnical design of anchor piles

A.11.5.1 Driven anchor piles

A.11.5.1.1 Basic considerations

Driven anchor piles can be designed to provide adequate capacity for both catenary and taut-leg mooring systems. The design of driven anchor piles builds on the strong industry background in the evaluation of geotechnical properties and the axial and lateral capacity prediction for driven piles, as developed and documented in this document. The recommended design criteria from ISO 19902 and from this document should be applied for the design of driven anchor piles, but with some modifications to reflect the differences between mooring anchor piles and fixed platform piles.

The design of a driven anchor pile should consider four potential failure modes:

a) pull-out due to axial forces;

b) overstress of the pile and mooring line attachment padeye due to lateral bending;

c) lateral rotation and/or translation;

d) fatigue due to environmental and installation actions.

In most anchor pile designs, the mooring line is attached to a padeye located on the pile below the seafloor to enhance the lateral capacity. As a result, the design should consider the mooring line angle at padeye connection resulting from the inverse catenary through the upper soil layers. Pre-tensioning is only required

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to develop the inverse catenary of the chain. Calculation of the soil resistance above the padeye location should also consider remoulding effects due to this trenching of the mooring line through the upper soil layers.

Driven anchor piles in soft clay typically have aspect ratios (penetration-to-diameter) of 25 to 30. Piles having such aspect ratios behave as if horizontally fixed in position at the pile tip, and consequently deflect laterally and fail in bending before translating laterally as a rigid body.

The design of driven anchor piles should consider typical installation tolerances, which can affect the calculated soil resistance and the pile structure. Pile verticality affects the angle of the mooring line at the padeye, which changes the components of horizontal and vertical mooring line forces that the pile is expected to resist. Underdrive affects the axial pile capacity and can result in higher bending stresses in the pile. Padeye orientation (azimuth) can affect the local stresses in the padeye and connecting shackle. Horizontal positioning can affect the mooring scope and/or angle at the vessel fairlead and should be considered when balancing mooring line pretensions.

A.11.5.1.2 Safety factors for driven anchor piles

Factors of safety for holding capacity of driven anchor piles are given in ISO 19901-7:2013, Table 7. Information on coupling between vertical and horizontal capacities can be found in A.11.5.2.2.5. Axial safety factors consider that the pile is primarily loaded in tension, and are therefore higher than for piles loaded in compression. As with other piled foundation systems, the calculated ultimate axial soil resistance should be reduced if soil set-up, which is a function of time after installation, is not complete before significant forces are imposed on the anchor pile.

As the lateral failure mode for piles is considered to be less catastrophic than the vertical mode, lower factors of safety are recommended for lateral pile capacity. Use of separate safety factors for vertical and lateral pile capacities can be straightforward for simple beam-column analysis [see A.11.5.2.2.3 item c)], but more complex methodologies do not differentiate between vertical and lateral pile resistance. The safety factor should be in accordance with the ISO 19901-7 criteria and the guidelines of A.11.5.2.2.5.

A.11.5.1.3 Basic considerations for structural strength design

The structural strength design for driven anchor piles should be based on the guidance provided in ISO 19902 and ISO 19901-7. Pile stresses should be limited by the provisions of ISO 19902 under ULS intact condition.

Anchor piles should be checked for fatigue caused by in-place mooring line forces. Fatigue damage due to pile driving stresses should also be calculated and combined with in-place fatigue damage. For typical mooring systems, fatigue damage due to pile driving is much higher than that caused by in-place mooring line forces.

Further guidance on fatigue damage design for driven piles can be found in References [103], [168] and [57].

A.11.5.1.4 Installation of driven anchor piles

Refer to 8.7.

A.11.5.2 Design of suction anchor piles

A.11.5.2.1 Basic considerations

Special care should be taken in the capacity, installation and retrieval assessments where sandy soils or sand layers are present. A suction anchor can take many forms, ranging from a gravity base with skirts to a no-ballast suction anchor that resists all applied actions by soil friction, lateral resistance and reverse end bearing (REB). Typically, the suction anchor will have a closed top, but if that is not the case the reverse end bearing can still be mobilised by the sum of inside skirt wall friction. It should be checked that the contribution from inside skirt wall friction do not exceed the reverse end bearing which can be the limiting factor after significant set up has occurred for a slender anchor.

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Generally, a suction anchor is technically feasible for soft to medium hard soils. For very soft soils, a suction anchor extends deep into the soil in order to reach competent bearing material. For very hard soils, it is sometimes not possible for the suction anchors to penetrate deep enough to provide adequate in-place strength. Some useful information for the design of suction anchors is provided in References [17], [107] and [163].

The design of suction anchors for floating systems includes the following aspects:

— penetration and removal;

— holding capacity;

— overstress of the pile and mooring line attachment padeye due to lateral bending;

— soil reactions or soil-structure interaction analyses for structural design.

In areas such as the Gulf of Mexico, where action effects of tropical cyclonic storms can exceed the capacity of the mobile mooring or mobile anchoring system, the design of suction piles should consider an anchor failure mode that reduces the chance of anchor pull-out. For site conditions where the presence of hard soil layers can limit suction anchor penetration, other anchor types should be considered instead.

The calculation of the representative holding capacity of the anchor should be based on the design value of the soil properties. Anchor adequacy with respect to installation should be checked against high estimate soil strength properties. If faced with larger-than-usual scatter in soil data, the designer should consider increasing the safety factors given in ISO 19901-7.

If the REB at the anchor tip is to be relied upon, it can possibly not be correct to add the representative value of the end bearing to the representative value of the skin friction to obtain the representative value of the axial capacity of the suction pile as the mobilization of REB can require large pile pullout displacements [84], [272]. Peak skirt wall friction is generally mobilised faster (less vertical displacement) than the reverse end bearing for suction anchors when the anchor is pulled out. Caution should therefore be made when calculating the ultimate axial holding capacity. Combining reverse end bearing based on a bearing capacity factor of N_c=9 works nicely with use of the peak skirt wall friction factor along the outside skirt wall [186].

The impact of the mooring line geometry in the soil on anchor forces should be considered since the geometry can affect the relationship between the horizontal and vertical anchor forces. The inverse catenary of the mooring line in the soil can make the mooring line angle steeper at the anchor padeye than at the seafloor. This steeper angle can result in a reduced horizontal force, but an increased vertical force at the anchor padeye. Both an upper and lower bound inverse catenary should be checked to ensure the worst-case anchor loading is established.

A.11.5.2.2 Analysis methods

A.11.5.2.2.1 Penetration analysis

A typical penetration analysis includes the calculation of three quantities for all penetration depths, which are:

— the penetration resistance exerted on the anchor by the soil;

— the required under-pressure to allow anchor embedment;

— the critical pressure that can cause the soil plug to fail.

The under-pressure ('suction') required for the pile to achieve design penetration should be properly estimated. Minimum under-pressures are vital input parameters to the structural design of the anchor. Furthermore, the pumps used during installation should be capable of generating adequate under-pressure.

a) Penetration resistance in clay

The penetration resistance can be calculated as the sum of the side shear and end bearing on the side wall and any other protuberances. Protuberances include mooring and lifting padeyes, longitudinal or ring stiffeners, changes in wall thickness, mooring chain, launching skids, and others. For an anchor

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in clay without protuberances and with a flat tip, the installation resistance at a given tip penetration depth, z, can be calculated by:

$$Q_{tot} = Q_{side} + Q_{tip} \quad \text{(A.60)}$$

$$Q_{side} = A_{wall} (\alpha_{ins} s_{uDSS})_{AVE} \quad \text{(A.61)}$$

$$Q_{tip} = (N_c s_{utip\ AVE} + \gamma' z) A_{tip} \quad \text{(A.62)}$$

where

Q_{tot}	is the total penetration resistance;
Q_{side}	is the resistance along the sides of the pile;
Q_{tip}	is the resistance at the pile tip;
A_{wall}	is the sum of inside and outside wall areas embedded in the soil;
A_{tip}	is the pile tip cross-sectional area (excluding contained soil);
α_{ins}	is the friction factor during installation [see item a)];
$\alpha_{ins}\, s_{uDSS}$	is the side friction;
$(\alpha_{ins}\, s_{uDSS})_{AVE}$	is the average side friction from seafloor to depth z;
N_c	is the bearing capacity factor [see item b)];
$s_{utip\ AVE}$	is the average of triaxial compression, triaxial extension, and DSS undrained shear strength at anchor tip level;
γ'	is the effective unit weight of soil;
z	is the tip penetration depth.

1) Friction factor during installation, α_{ins}

The friction factor during installation, α_{ins}, is usually defined as the ratio of remoulded shear strength over undisturbed shear strength, which is as the inverse of the soil sensitivity. The friction factor can be determined by various methods for which fall cone, UU triaxial and miniature vane (minivane) are the most common. The typical range of α_{ins} for soft clays is 0,2 to 0,5.

There can be uncertainty in the soil sensitivity since it is influenced by the quality of the intact strength that it is related to. Alternatively, the side friction, $\alpha_{ins} s_{uDSS}$, can be equated to the direct measurement of remoulded shear strength through fall cone, UU triaxial or minivane tests. The remoulded strength used in design should reflect both the directly measured value and the value derived from the intact strength divided by the sensitivity.

Some installation records have shown that the interface shear strength mobilised during installation can, at a given depth, be less than $\alpha_{ins} s_{uDSS}$. In cases where the full interface shear strength, $\alpha_{ins} s_{uDSS}$, cannot be mobilised along the anchor wall, such as when the anchor is painted or subjected to unusual surface treatment, a correction factor should be applied to α_{ins} to properly predict the penetration resistance [17], [59], [99]. Ring shear tests, with the actual wall surface modelled in the tests, can also be used to measure the actual interface shear strength.

2) Bearing capacity factor, N_c

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The value of the bearing capacity factor, N_c, to be used to calculate the penetration resistance of the anchor tip or of a given protuberance depends on the shape of the protuberance and the ratio of the width of the protuberance over the embedment depth of the protuberance. Values of N_c ranging from 5,1 to 9,0 for strip and circular footings are recommended in Reference [308].

Because the anchor wall thickness is usually small compared to the anchor diameter and the embedment depth, the pile tip is usually considered to be a deeply embedded strip footing with an associated bearing capacity factor, N_c, equal to 7,5. The values of N_c to be used in Formula (A.62) are summarized in Table A.13.

A detailed example of the calculation of N_c is given in Reference [116]. Values of N_c different from those of Table A.13 are acceptable provided that they can be documented by appropriate modelling and test results.

Table A.13 — Recommended N_c factor

Purpose	Shape or area	N_c
Calculation of pile tip penetration resistance	Strip	7,5
Calculation of critical under-pressure causing soil plug failure [see A.11.5.2.2.1 item c)]	Circular	6,2 to 9,0 depending on embedment ratio [308]
Calculation of penetration resistance of protuberances [see A.11.5.2.2.1 item 3)]	Varies	5,0 to 13,5 [249]

3) Changes in penetration resistance due to protuberances

Formulae (A.60) to (A.62) should be modified if protuberances are present. The change in penetration resistance due to the presence of mooring and lifting padeyes, longitudinal or ring stiffeners, mooring chain, launching skids, pile tip other than flat (i.e. bevelled) or any other internal or external protuberance should be addressed to assess the changes in friction and end bearing resistance caused by the protuberances. Most protuberances cause an increase in penetration resistance, except for internal ring stiffeners which can cause a decrease in internal side friction if they are closely spaced [17], [59].

b) Required under-pressure

The required under-pressure, ΔU_{req}, to embed the anchor can be calculated as follows:

$$\Delta U_{req} = (Q_{tot} - W') / A_{in} \quad \text{(A.63)}$$

where

Q_{tot} is the total penetration resistance;

W' is the submerged weight of the anchor during installation;

A_{in} is the plan view inside area where under-pressure is applied.

c) Critical and allowable under-pressures

The critical under-pressure at a given depth, ΔU_{crit}, defined as the under-pressure that causes a general reverse end bearing failure at the anchor tip and large soil heave within the anchor, can be calculated at a given depth as follows:

$$\Delta U_{crit} = N_c s_{utip\ AVE} + [A_{inside} (\alpha_{ins} s_{uDSS})_{AVE}] / A_{in} \quad \text{(A.64)}$$

where A_{inside} is the inside lateral area of anchor wall.

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In shallow water, the critical under-pressure should not exceed the water cavitation pressure.

The recommended allowable under-pressure, ΔU_{allow}, defined as the maximum under-pressure to be applied to the anchor, can be calculated as the critical under-pressure divided by an appropriate safety factor. The minimum value of the safety factor is typically 1,5. Lower values can be acceptable provided that the soil plug behaviour is monitored during installation and it is confirmed that no plug failure occurred, and provided that the calculated allowable under-pressure is acceptable for the pile steel structure (i.e. no risk of buckling).

d) Soil plug heave inside anchor

The soil heave inside the anchor during installation can be estimated by assuming that a percentage of the soil volume displaced by the cross-sectional area of the anchor goes inside the anchor. This percentage depends on anchor tip geometry and mode of penetration (i.e. self-weight penetration versus penetration by under-pressure). It is commonly assumed that 50 % of the soil displaced by the cross-sectional area of the anchor tip goes inside the anchor during self-weight penetration if the anchor tip is flat.

The final elevation of the internal plug surface depends on the wall thickness variations, internal soil plug stability, and spacing and type of any internal stiffeners [8].

Soil heave should be accounted for in calculating the required pile stick-up and total length.

e) Penetration in sand

Procedures for calculating skirt penetration resistance of skirted foundations in sand can be found in References [14], [167] and [304].

The skirt penetration resistance in sand consists of the two components, skirt wall friction and tip resistance, as for clay. The bearing capacity and skirt wall friction have typically been calculated by two different models:

1) Bearing capacity approach

In the bearing capacity model, the penetration resistance is calculated as;

$$P_f = Q_{tip} + Q_{side} = q_{tip} A_{tip} + f_{s,av} A_{wall} \quad \text{(A.65)}$$

where

q_{tip} is $0{,}5\gamma' t N_{\gamma} + q N_q$;

$fs_{,av}$ is $0{,}5\ \gamma' z K \tan\delta$ (average friction over skirt length);

A_{tip} is $\pi D t$;

A_{wall} is $2Dz$ (sum of inside and outside);

Nq is $e^{\pi \tan\varphi} \cdot \tan^2(45+\varphi/2)$. $N_{\gamma} = 1{,}5(Nq-1)\tan\varphi$;

Q is the effective overburden pressure outside skirt at skirt tip level;

γ' is the effective unit weight of sand;

φ is the peak drained friction angle of sand (angle based on a reference vertical effective consolidation stress of σ'_{vc}=200 kPa);

r is the roughness factor between skirt wall and sand (r=0,9 is suggested used);

δ is the friction angle between sand and skirt wall $\delta = r\varphi$;

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K is the ratio between horizontal and initial vertical effective stresses at skirt wall (K=0,8 is suggested used);

T is the skirt wall thickness (assumed to be small compared to the diameter);

D is the diameter of foundation or anchor (external D can be used since t is small);

z is the depth below sand surface.

Formula (A.65) for skirt wall friction assumes homogeneous sand conditions. For a horizontally layered soil profile the wall friction can be calculated as the sum of the contributions from the different soil units. For foundations or anchors with long skirt walls, the skirt wall friction can give an important additional vertical stress outside the skirt wall. This increase can cause an increased skirt tip resistance. With reference to Reference [14], this contribution can be added to the in situ effective overburden;

$$q = \gamma' z + \alpha_f f_{s,tip} = \gamma' z\left(1 + \alpha_f K tan\delta\right) \quad \text{(A.66)}$$

where

α_f is the ratio between vertical normal stress increase and skirt wall friction at skirt tip level (α_f=1 suggested);

$f_{s,tip}$ is the skirt wall friction at skirt tip level.

2)Empirical CPT resistance approach

In the CPT resistance approach, the penetration resistance is linked to the measured cone tip resistance as follows:

Pf is $Q_{tip} + Q_{side} = k_{tip} qc A_{tip} + A_{wall} \int k_{side} q_c(z)\, dz$;

q_c is the cone tip resistance measured in CPT tests;

K_{tip} is the empirical skirt tip constant related to q_c;

K_{side} is the empirical skirt wall friction constant related to q_c

As presented in Reference [14], the empirical constants derived from back calculations of skirts penetrated by self-weight only may give a large range as illustrated below:

K_{tip} = 0,01 – 0,55 when k_{side} = 0,001 5;

K_{tip} = 0,03 – 0,60 when k_{side} = 0,001 0.

Empirical constants can also be found in Reference [104]. These are also factors suggested for skirt penetration without the use of under-pressure.

A CPT based method for installation of suction caissons in sand is suggested in Reference [304].

f) Penetration in sand- effect of under-pressure

By applying under-pressure in a sand without silt or clay layers, the under-pressure will cause seepage flow of water from the outside to the inside skirt compartment. This flow will change the effective stresses in the sand with an increase along the outside skirt wall while beneath the skirt tip and along the inside skirts the effective stresses will decrease. In total, this will result in a significantly reduced penetration resistance where approximately all the tip resistance and the skirt wall friction along the inside skirt wall approach zero. The maximum achievable penetration depth in a clean sand can simplified be assumed equal to the anchor diameter (z/D = 1) [167]. This assumes a typical submerged weight of the anchor whilst a very high submerged weight can increase the possible penetration depth. More details on critical and allowable under-pressure can be found in References [14] and [167]. With

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