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ISO 19901-4

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

Part 4: Geotechnical and foundation design considerations

Industries du pétrole et du gaz naturel — Exigences spécifiques relatives aux structures en mer —

Partie 4: Bases conceptuelles des fondations

Reference number
ISO 19901-4:2016(E)

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Foreword

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

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

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

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

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

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

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

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

— *Part 1: Metocean design and operating considerations*

— *Part 2: Seismic design procedures and criteria*

— *Part 3: Topsides structure*

— *Part 4: Geotechnical and foundation design considerations*

— *Part 5: Weight control during engineering and construction*

— *Part 6: Marine operations*

— *Part 7: Stationkeeping systems for floating offshore structures and mobile offshore units*

— *Part 8: Marine soil investigations*

The following part is under preparation:

— *Part 9: Structural integrity management*

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

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

— ISO 19901 (all parts), *Petroleum and natural gas industries — Specific requirements for offshore structures*

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

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

— ISO 19904, *Petroleum and natural gas industries — Floating offshore structures*

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

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

— ISO 19905-3, *Petroleum and natural gas industries — Site specific assessment of mobile offshore units — Part 3: Floating units (under preparation)*

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

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

— ISO 13623 (all parts), *Petroleum and natural gas industries — Pipeline transportation systems*

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

Introduction

The International Standards for offshore structures, ISO 19900 to ISO 19906, constitute a common basis covering those aspects that address design requirements and assessments of all offshore structures used by the petroleum and natural gas industries worldwide. Through their application, the intention is to achieve reliability levels appropriate for manned and unmanned offshore structures, whatever the type of structure and the nature of the materials used.

It is important to recognize that structural integrity is an overall concept comprising models for describing actions, structural analyses, design rules, safety elements, workmanship, quality control procedures and national requirements, all of which are mutually dependent. The modification of one aspect of design in isolation can disturb the balance of reliability inherent in the overall concept or structural system. The implications involved in modifications, therefore, need to be considered in relation to the overall reliability of all offshore structural systems.

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

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

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

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

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

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

— 'shall' and 'shall not' are used to indicate requirements strictly to be followed in order to comply with the document and from which no deviation is permitted;

— 'should' and 'should not' are used to indicate that among several possibilities one is recommended as particularly suitable, without mentioning or excluding others, or that a certain course of action is preferred but not necessarily required, or that (in the negative form) a certain possibility or course of action is deprecated but not prohibited;

— 'may' and 'need not' are used to indicate a course of action permissible within the limits of the document;

— 'can' and 'cannot' are used for statements of possibility and capability, whether material, physical or causal.

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

Part 4: Geotechnical and foundation design considerations

1 Scope

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

— site and soil characterization;

— identification of hazards;

— design and installation of shallow foundations supported by the seabed;

— design and installation of pile foundations;

— soil-structure interaction for auxiliary structures, e.g. subsea production systems, risers and flowlines (guidance given in A.10);

— design of anchors for the stationkeeping systems of floating structures (guidance given in A.11).

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

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

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

2 Normative references

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

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

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

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

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

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

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

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

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

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

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

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

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

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

3 Terms and definitions

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

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

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

[SOURCE: ISO 19900:2013, 3.1]

3.2
action factor
partial safety factor applied to a design action

3.3
basic variable
one of a specified set of variables representing physical quantities which characterize actions, environmental influences, geometric quantities, or material properties including soil properties

[SOURCE: ISO 19900:2013, 3.7]

3.4
characteristic value
value assigned to a basic variable associated with a prescribed probability of not being violated by unfavourable values during some reference period

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

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

— an upper value with an intended probability of not being exceeded or a lower value with an intended probability of being achieved, during some specific reference period;

— a nominal value, which may be specified in cases where a statistical distribution is not known.

[SOURCE: ISO 19900:2013, 3.10]

3.5
design actions
combination of representative actions and partial safety factors representing a design situation for use in checking the acceptability of a design

3.6
design value
value derived from the representative value for use in the design verification procedure

[SOURCE: ISO 19900:2013, 3.18]

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

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

3.8
effective foundation area
reduced foundation area having its geometric centre at the point where the resultant action vector intersects the foundation base level

3.9
limit state
state beyond which the structure no longer satisfies the relevant design criteria

[SOURCE: ISO 19900:2013, 3.2]

3.10
material factor
partial safety factor applied to the characteristic strength of the soil, the value of which reflects the uncertainty or variability of the material property

Note 1 to entry: See ISO 19900.

3.11
representative value
value assigned to a basic variable for verification of a limit state

[SOURCE: ISO 19900:2013, 3.38]

3.12
resistance
capacity of a component, or a cross-section of a component, to withstand action effects without failure

[SOURCE: ISO 19900:2013, 3.39]

3.13
resistance factor
partial safety factor applied to the characteristic capacity of a foundation, the value of which reflects the uncertainty or variability of the component resistance including those of material property

3.14
scour
removal of seabed soils caused by currents, waves and ice

[SOURCE: ISO 19900:2013, 3.43]

3.15
seabed
materials below the seafloor, whether of soils such as sand, silt or clay, cemented materials or of rock

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

3.16
seafloor
interface between the sea and the seabed

3.17
serviceability
ability of a structure or structural member to perform adequately for a normal use under all expected actions

[SOURCE: ISO 2394:2015, 2.1.32]

3.18
settlement
permanent downward movement of a structure as a result of its own weight and other actions

3.19
strength
mechanical property of a material indicating its ability to resist actions, usually given in units of stress

Note 1 to entry: See ISO 19902.

3.20
undrained condition
condition whereby the applied stresses and stress changes are supported by both the soil skeleton and the pore fluid and do not cause a change in volume

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

4 Symbols and abbreviated terms

4.1 General

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

4.2 Symbols for shallow foundations design

A actual (cross-sectional plan) foundation area

A' effective foundation area depending on eccentricity of actions

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

A_p projected area of skirt tip

A_s side surface area of skirt embedded at a particular penetration depth

$A_{idealized}$ idealized rectangular foundation area, for irregular foundation shapes

b_c, b_q, b_γ bearing capacity correction factors related to foundation base inclination

B minimum lateral foundation dimension (also foundation width)

B' minimum effective lateral foundation dimension (also foundation effective width)

C compression index of soil over loading range considered

d_c, d_q, d_γ bearing capacity correction factors related to foundation embedment depth

D foundation diameter (for circular foundations)

D_b depth below seafloor to foundation base level

e eccentricity of action

e_0 initial void ratio of the soil

e_1 eccentricity of action in coordinate direction 1

e_2 eccentricity of action in coordinate direction 2

f unit skin friction resistance along foundation skirts during installation

F bearing capacity correction factor to account for undrained shear strength heterogeneity

g_c, g_q, g_γ correction factors related to seafloor inclination

G elastic shear modulus of soil

h soil layer thickness

H horizontal action

H_b horizontal action on effective area component of the base

H_d design value of resistance to pure sliding

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

H_{ult} ultimate horizontal capacity in yield surface design method

i_c, i_q, i_γ bearing capacity correction factors related to foundation action inclination

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

K_p coefficient of passive earth pressure

K_{rd} drained horizontal soil reaction coefficient

K_{ru} undrained horizontal soil reaction coefficient

L maximum lateral foundation dimension (also foundation length)

L' maximum effective lateral foundation dimension (also foundation effective length)

M overturning moment

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

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

4.3 Symbols for pile foundations design

A_{pile} gross end area of pile, $A_{pile} = \dfrac{\pi \cdot D^2}{4}$

A_r pile displacement ratio, $A_r = \dfrac{A_w}{A_{pile}} = 1 - \left(\dfrac{D_i}{D}\right)^2$

A_w cross-sectional area of pile annulus, $A_w = \dfrac{\pi}{4} \cdot \left(D^2 - D_i^2\right)$

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, $D_i = D - 2\ WT$

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 = $L - z$

J dimensionless empirical constant, for p–y curves for clay

k initial modulus of subgrade reaction, for p–y curves for sand

K_0 coefficient of lateral earth pressure at rest

L embedded length of pile below original seafloor

L_s length of soil plug in sand layers

N_q dimensionless bearing capacity factor

p mobilized lateral capacity, for p–y curves

p_a atmospheric pressure (p_a = 100 kPa)

$P_{d,e}$ design value of axial action on the pile, determined from a coupled linear structure and non-linear foundation model using the design actions for extreme conditions

$P_{d,p}$ design value of axial action on the pile, determined from a coupled linear structure and non-linear foundation model using the design actions for permanent and variable actions or the design axial action for operating situations

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

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

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

$p'_m(z)$ *in situ* effective mean stress at depth z

P_o pile outer perimeter = πD

q unit end bearing at pile tip

$q_c (z)$ CPT cone resistance at depth, z, in stress units

$q_{c,f} (z)$ reduced CPT cone resistance at depth, z, to account for general scour

$q_{c,av,1,5D}$ average value of $q_c (z)$ between 1,5 D above pile tip and 1,5 D below pile tip

$q_{c,tip}$ CPT cone resistance at pile tip

Q mobilized end bearing capacity in Q–z curves

$Q_{f,c}$ skin friction capacity in compression

$Q_{f,t}$ skin friction capacity in tension

$Q_{f,i,clay}$ cumulative skin friction capacity of clay layers within soil plug, for CPT-based method 3

Q_{lim} limiting unit end bearing value

Q_p end bearing capacity

Q_r representative value of axial pile capacity

$Q_{r,c}$ representative value of axial pile capacity in compression

$Q_{r,t}$ representative value of axial pile capacity in tension

s_u undrained shear strength

$s_u(z)$ undrained shear strength at depth z

WT pile wall thickness

t mobilized skin friction for axial shear transfer t–z curves

t_{max} maximum skin friction for axial shear transfer t–z curves

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

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

D flowline, or pipeline, diameter

f_c dimensionless cyclic factor

f_t dimensionless time factor

f_V dimensionless velocity factor

G_{max} initial elastic (small strain) shear modulus of soil

H lateral (horizontal) soil resistance

K_{max} maximum value of normalized secant stiffness on initial unloading or reloading

I_P plasticity index of soil

k_v secant stiffness of equivalent spring = $\Delta Q / \Delta z$

N integrated normal contact force

N_c dimensionless bearing capacity factor

Q_{smax} maximum suction (uplift) force, per unit length of pipeline

Q_u limiting penetration resistance, per unit length of pipeline

s_u undrained shear strength

s_{uDSS} undrained shear strength obtained in direct simple shear mode

s_{ur} remoulded undrained shear strength

T drained axial resistance per unit length of pipeline

V vertical action on pipeline

z depth to flowline, or pipeline, invert

ΔQ change in vertical force, per unit length of pipeline

Δz change in vertical displacement

Δz_b uplift (break-out) displacement

δ interface friction angle at soil–pipeline interface

μ pipeline–soil friction coefficient

ζ dimensionless enhancement factor

ζ_t dimensionless time factor

ζ_V dimensionless velocity factor

$\theta_D{}'$ half-angle of pipeline–soil contact perimeter

4.5 Symbols for design of anchors for stationkeeping systems

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

S_t soil sensitivity

s_u undrained shear strength at the point in question

$s_{u,AVE}$ average undrained shear strength within the failure zone at the design penetration depth, corrected for effects of cyclic loading

$s_{u,tip\ AVE}$ average of triaxial compression, triaxial extension, and DSS undrained shear strength at anchor tip penetration depth

$s_{u,DSS}$ undrained shear strength obtained from direct simple shear tests

t time

t_r time of anchor retrieval

v free-fall velocity of a gravity-embedded anchor

V vertical action component

W_s submerged weight of a gravity-embedded anchor

W' submerged weight of anchor

W'_{cw} submerged unit weight of chain or wire rope

z embedment or penetration depth

ΔU_{req} required underpressure to embed a suction anchor pile

ΔU_{crit} critical underpressure causing failure of soil plug inside a suction anchor pile

α_{ins} friction factor during installation of a suction anchor pile or of a gravity-embedded anchor

γ' submerged unit weight of soil

η empirical reduction factor accounting for progressive failure of a plate anchor

ρ fluid density

θ angle of mooring line at anchor padeye attachment point (measured from horizontal)

θ_{axial} angle of mooring line at anchor padeye attachment point (measured from horizontal) above which the anchor ultimate capacity is controlled by axial capacity

$\theta_{lateral}$ angle of mooring line at anchor padeye attachment point (measured from horizontal) below which the anchor ultimate capacity is controlled by lateral capacity

4.6 Abbreviated terms

BOP blow-out preventer

CPT cone penetration test

FEM finite element method

FOS factor of safety

PFD partial factor design

REB reverse end bearing

SCR steel catenary riser

SLS serviceability limit state

SRD soil resistance to driving

TDP touch-down point

UHC ultimate holding capacity

ULS ultimate limit state

VLA vertically loaded anchor

5 General requirements

5.1 General

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

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

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

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

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

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

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

5.2 Design cases and safety factors

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

— ISO 19902 for fixed steel offshore structures;

— ISO 19903 for fixed concrete offshore structures;

— ISO 19904 and ISO 19901-7 for floating offshore structures;

— ISO 19905-1 for jack-ups;

— ISO 19906 for arctic offshore structures.

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

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

5.3 Characteristic values of soil properties

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

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

— the definition used for the characteristic value;

— the assumptions made in the calculation model;

— the amount and quality of site investigations and possible environmental factors, including insufficient data and imprecise knowledge;

— a priori knowledge such as geological information and physically credible values;

— the measurable physical quantities that correspond to, and are representative of, the population of the properties considered in the calculation model;

— the appropriate factors or transformation functions, to convert the properties obtained from laboratory and/or *in situ* tests or other methods to properties corresponding to the assumptions made in the calculation model;

— the measurement error, conversion factor uncertainty and statistical uncertainty;

— the spatial variability of soil within each stratum;

— other variance reduction by appropriate methods (if relevant and if enough data are available).

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

5.4 Testing and instrumentation

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

Possible testing and instrumentation methods include the following:

a) loading tests and/or model tests

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

— the foundation configuration differs significantly from earlier configurations where operational experience exists;

— the soil conditions differ significantly from those where operational experience exists;

— new methods of installation or removal are envisaged;

— a high degree of uncertainty exists as to how the structure or its foundation will behave.

b) temporary instrumentation

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

— the installation method presupposes the existence of measured data for control of the operation;

— an installation method is to be applied with which little or no experience has been gained.

c) permanent instrumentation

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

— the safety or behaviour of the foundation is dependent on active operation;

— the foundation configuration, the soil conditions, or the actions differ substantially from those with which experience has been gained;

— there is a need for monitoring the whole foundation with regard to penetration, settlement, tilt, or other behaviour;

— the method of removal presupposes the existence of measured data for control of the operation.

6 Geotechnical data acquisition and identification of hazards

6.1 General

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

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

6.2 Shallow geophysical investigation

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

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

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

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

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

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

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

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

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

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

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

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

6.3 Geological modelling and identification of hazards

6.3.1 General

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

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

Some of the more familiar geological processes, events and conditions are discussed in 6.3.2 to 6.3.7.

6.3.2 Earthquakes

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

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

6.3.3 Fault planes

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

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

6.3.4 Seafloor instability

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

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

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

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

6.3.5 Scour and sediment mobility

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

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

a) local scour

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

b) global scour

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

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

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

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

6.3.6 Shallow gas

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

6.3.7 Seabed subsidence

The nature of the soil conditions and the reservoir and extraction processes should be investigated to establish whether subsidence of the seabed is likely to occur during the field life. The potential for seabed subsidence, where this is a possibility due to reservoir compaction, should be addressed by considering an increase in air gap. The magnitude of surface settlements will depend on the reservoir dimensions, compressibility of reservoir rocks and the anticipated pressure drop.

6.4 Carbonate soils

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

7 Design of shallow foundations

7.1 General

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

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

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

The following is noted.

— It is primarily intended that these recommendations be applied to the design of temporary foundations, such as jacket mudmat foundations with shallow skirts used for temporary support during installation, under idealized conditions.

— The methods presented are intended for combinations of dead action, variable live action and environmental action (i.e. wind, wave and current); where these arise from transient or cyclic actions they are considered as quasi-static actions. Specialist advice shall be sought where more complex dynamic analysis is required, such as where the inertia of the structure or foundation soils is important (e.g. seismic loading).

— These recommendations can also be considered for foundations that are to be used for permanent support. This is applicable only where permanent foundations are subject to simple loading and soil conditions, for which the recommendations remain valid; or for situations where foundation failure would result in displacements and rotations that could be shown to have minimal environmental, safety and economic consequences.

— Alternative design approaches can be more applicable for design of shallow foundations used in more complicated and critical situations, such as complex seabed conditions or complex multi-directional cyclic actions (including cases in which cyclic uplift actions can largely offset the foundation dead weight). Advanced analysis techniques such as finite element modelling can also be used to confirm that design requirements are met for more complex situations. Ultimate limit state under uplift requires special attention with due regard to negative pore pressures (suction), adhesion, permeability of the soil, drainage paths, duration of the action and geometry of the foundation. Unless other failure modes are more critical, such as where separation of the foundation from the seabed occurs due to drainage, uplift resistance can be analysed as a reverse bearing mechanism applying the relevant formulae for general shear failure in compression presented below and in A.7.

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

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

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

— care should be taken to appropriately transfer the seafloor actions to skirt tip level, as described in A.7.2.3;

— when assessing stability based on loading at skirt tip level, it is important to ensure that the penetration is sufficient to avoid internal failure mechanisms forming within the skirted compartment, which might lower the overall capacity. Required penetration into the stronger soil to achieve this would typically be between 1/10 and 1/5 of the spacing between skirts, although this depends on the site specific soil profile and foundation action. A case-specific analysis is required to confirm an acceptable penetration;

— the extent of skirt penetration should be assessed. Additional geotechnical/structure assessment is required in the event full skirt penetration is unachievable and no measure is taken to ensure full penetration.

7.2 Principles

7.2.1 General principles

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

— bearing failure constitutes any failure mode that could result in excessive combinations of vertical displacement, lateral displacement, or overturning rotation of the foundation; while pure sliding or torsional failure corresponds to a failure mode where the foundation translates or twists only in a horizontal plane;

— foundation stability can be analysed by limit equilibrium methods ensuring equilibrium between design actions and design resistance. The shape and location of the critical failure mechanism shall be determined, which will depend on the design actions, soil stratification and foundation geometry. Due consideration shall be given to the possibilities of excessive displacement and deformation of the foundation soil, and where these are critical, more complex analysis approaches are required;

— calculations using alternative methods of analysis shall include an explanation of any possible differences due to the method adopted;

— design actions shall be determined with due consideration of the design life of the foundation;

— seafloor gradient and/or installation tolerance shall be taken into account in design. Tolerable foundation tilt should be specified;

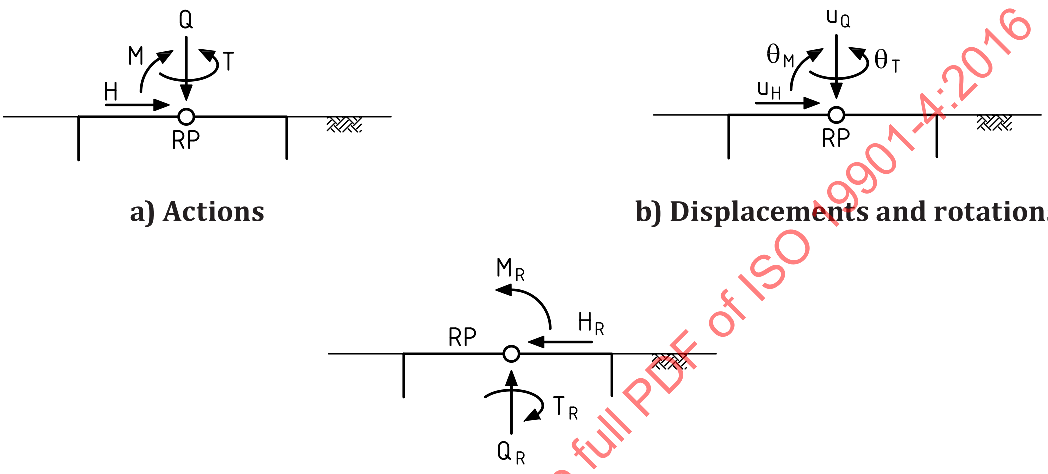
— undrained calculations shall be adopted where no drainage, and hence no dissipation of excess pore pressures, occurs during loading. This can occur as a result of the rate of loading or the impermeable nature of the soil. In contrast, drained calculations shall be adopted where no excess pore pressures arise during loading. Analysis of foundations subject to partial soil drainage during the loading event is complex, and specialist advice shall be sought in these cases;

— design can be based on serviceability (rather than stability) criteria, whereby the deformation of the foundation is assessed against allowable movement criteria. The appropriateness of adopting this approach will depend on the type of structure and its installation. The selection of appropriate soil moduli (especially considering strain dependency) is essential in calculation of serviceability limit states;

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

7.2.2 Sign conventions, nomenclature and action reference point

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



a) Actions

b) Displacements and rotations

c) Seabed resistance

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

7.2.3 Action transfer

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

7.2.4 Idealization of foundation area and the effective area concept

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

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

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

7.3 Acceptance criteria and design considerations

7.3.1 Action and material factors

When calculating stability:

— the soil strength shall be determined using a material factor γ_m = 1,25 except as indicated below or in A.7. Where geotechnical data are sparse or site conditions are particularly uncertain, or where high uncertainty exists in relation to potential failure mechanisms or methods of analysis, increasing the above material factor can be warranted. In accordance with ISO 19900, a material factor of 1,0 is permitted for accidental loading;

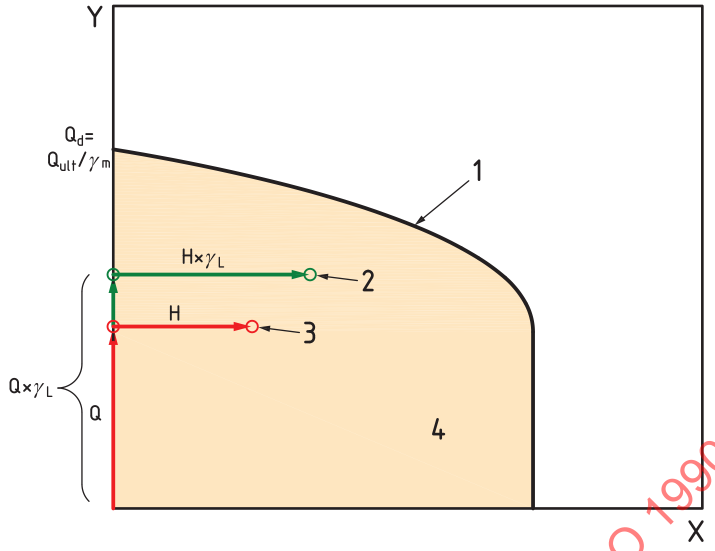
— partial action factors shall be determined based on guidance from the relevant standard of the ISO 19900- series. The weight of the soil, including that within the skirts, should normally be calculated with factors equal to unity (see 7.3.4.1). In some situations, an action factor below or above 1,0 can be justified in order to ensure a sufficiently robust design;

— partial action factors and material factors shall be applied with consistency throughout the design process. In particular, note that the ratio between the resultant horizontal action and the resultant vertical action on the foundation will influence many of the design formulae provided in A.7.

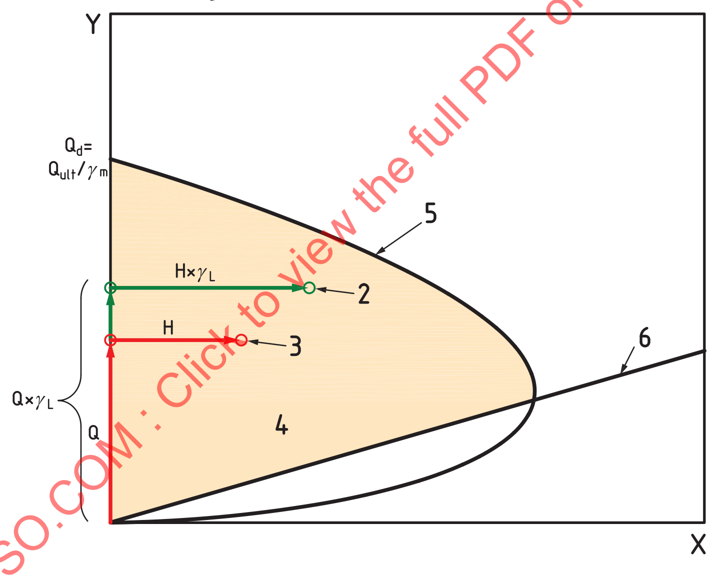
When calculating displacements as part of serviceability based design, all partial action and material factors should generally be set to unity. However, where foundation displacement can lead to unacceptable consequences or govern design, it can be appropriate to apply factors other than unity in order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

7.3.2 Use in design

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



a) Undrained conditions



b) Drained conditions

Key

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

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

7.3.3 Special cases

For assessment of foundation stability during set-down on the seabed, stability can be assessed based on the applied vertical actions only. In this case, the ultimate limit state shall be calculated using a material factor of γ_m = 1,5. The increased material factor in this case leads to an increased margin of safety against bearing failure during set-down and assists in settlement and/or embedment of the foundation into the seabed remaining modest.

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

7.3.4 Additional design considerations

7.3.4.1 Adjusting for soil plug weight in skirted foundations

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

— the soil height above skirt tip level can be higher inside the skirt than outside the skirt, such as where significant scour has occurred, or

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

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

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

where

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

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

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

A is the actual cross-sectional plan foundation area.

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

7.3.4.2 Horizontal seabed resistance above foundation base level

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

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

The additional horizontal resistance, ΔH_{d}, that can be mobilized between seafloor and foundation base level can be approximately calculated from Formulae (2) and (3).

Total horizontal sliding resistance is given by $H_{\text{d}} + \Delta H_{\text{d}}$.

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

$$\Delta H_{\mathrm{d}} = K_{\mathrm{ru}} \left(\frac{s_{\mathrm{u,ave}}}{\gamma_{\mathrm{m}}} \right) A_{\mathrm{h}} \tag{2}$$

where

ΔH_{d} is the horizontal soil resistance due to active and passive earth pressures on foundation skirts;

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

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

γ_{m} is the material factor (see 7.3.1);

A_{h} is the vertical projected area of the foundation in the direction of sliding.

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

$$\Delta H_{\mathrm{d}} = K_{\mathrm{rd}} (0{,}5 \gamma ' D_{\mathrm{b}}) A_{\mathrm{h}} \tag{3}$$

where

ΔH_{d} is the additional horizontal resistance mobilized between the seafloor and foundation base level;

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

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

D_{b} is the depth to base level;

A_{h} is the vertical projected area of the foundation in the direction of sliding.

7.3.4.3 Shallow foundations penetrating into soft soils

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

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

7.3.4.4 Tensile stresses beneath foundation

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

Skirted foundations (except those with perforated mudmats) can resist transient tension through generation of negative excess pore pressures between the confined soil plug and underside of the foundation top cover. Cyclic tensile stresses (relative to ambient water pressure) from waves with a few seconds duration can often be demonstrated as acceptable in design; while longer duration tensile stresses can potentially be carried by skirted foundations on clays with low permeability. Specialized advice shall be sought in relation to reliance on tensile stresses beneath a foundation.

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

7.3.4.5 Non-standard soils or soil profiles

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

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

7.3.4.6 Interaction with other structures

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

7.3.4.7 Multiple foundations

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

7.3.4.8 Consideration of surrounding seabed conditions

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

7.3.4.9 Carbonate soils

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

Further discussion of carbonate soils is presented in 6.4 and A.6.4 with particular reference to shallow foundations in A.6.4.4.3.

7.3.5 Alternative method of design based on yield surfaces

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

7.3.6 Selection of soil parameter values for design

7.3.6.1 Shear strength used in stability analysis

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

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

— imprecise definition of the characteristic values of soil strength parameters used in the analysis methods;

— variability in strength measurements (which depends on the scope and content of the soil investigation, sampling disturbance, methods of testing, etc.);

— the strategies, methods, and procedures used for assessment of the characteristic value of strength, which vary from one designer or design environment to another;

— the amount of testing forming the basis for proposed characteristic values of shear strengths, statistical variability, and bias.

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

7.3.6.2 Parameters used in serviceability design

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

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

7.4 Stability of shallow foundations

7.4.1 Assessment of bearing capacity

7.4.1.1 Undrained conditions (constant shear strength with depth)

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

$$q_\mathrm{d} = N_\mathrm{c} \frac{s_\mathrm{u}}{\gamma_\mathrm{m}} K_\mathrm{c} \qquad (4)$$

where

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

N_c is the undrained bearing capacity factor, equal to 5,14;

s_u is the characteristic value of undrained shear strength of the soil;

γ_m is the material factor (see 7.3);

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

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

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

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

— Infinitely long strip foundation

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

— Circular or square foundation

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

7.4.1.2 Undrained conditions (linearly increasing shear strength with depth)

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

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

where

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

F is a correction factor given as function of $\kappa B'/s_{u0}$;

N_c is the undrained bearing capacity factor, equal to 5,14;

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

κ is the rate of increase of the characteristic value of undrained shear strength with depth;

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

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

γ_m is the material factor (see 7.3).

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

7.4.1.3 Drained conditions

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

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

where

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

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

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

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

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

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

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

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

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

— Infinitely long strip foundation

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

— Circular or square foundation

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

7.4.2 Assessment of sliding capacity

7.4.2.1 General

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

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

7.4.2.2 Undrained conditions

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

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

where

H_d is the design resistance for pure sliding;

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

γ_m is the material factor (see 7.3);

A is the actual cross-sectional plan foundation area.

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

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

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

7.4.2.3 Drained conditions

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

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

where

H_d is the design resistance of sliding;

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

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

γ_m is the material factor (see 7.3).

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

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

7.4.3 Assessment of torsional capacity

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

Effects of torsion on foundation stability can be considered through a yield surface approach, see A.7.

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

7.5 Serviceability (displacements and rotations)

7.5.1 General

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

7.5.2 Displacement under static loading

7.5.2.1 General

Calculation of foundation displacements should include:

— immediate displacements;

— primary consolidation settlement;

— secondary compression (creep) settlement;

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

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

7.5.2.2 Immediate displacements

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

Vertical

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

Horizontal

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

Overturning

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

Torsion

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

where

u_Q is the vertical displacement at foundation base level;

u_H is the horizontal displacement at foundation base level;

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

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

Q is the vertical action;

H is the horizontal action;

M is the overturning moment;

T is the torsional moment;

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

ν is the Poisson's ratio of the soil;

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

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

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

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

Numerical analysis methods should be considered for more complex situations.

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

7.5.2.3 Primary consolidation settlement

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

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

where

u_Q is the vertical displacement at foundation base level;

h is the layer thickness;

e_0 is the initial void ratio of the soil;

C is the characteristic value of compression index of the soil over the loading range considered;

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

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

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

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

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

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

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

7.5.2.4 Secondary compression (creep)

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

7.5.2.5 Differential settlements induced by eccentricity

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

7.5.3 Displacement under dynamic and cyclic actions

7.5.3.1 Foundation response to applied loading

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

7.5.3.2 Settlement after the event

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

7.5.4 Other contributors to foundation settlement

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

7.6 Other design considerations

7.6.1 Hydraulic stability

7.6.1.1 Scour

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

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

— placing scour-resistant materials around the edges of the foundation.

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

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

7.6.1.2 Piping

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

7.6.2 Installation, retrieval and removal

7.6.2.1 General

Installation shall be planned to ensure that the foundation can be properly seated at the intended site without unintended or excessive disturbance to the supporting soil. Penetration of foundation skirts into the seabed can be facilitated by providing under-pressure (relative to the ambient hydrostatic pressure) inside the skirt compartments under the foundation. In the case of providing under-pressure, installation procedures shall be planned to avoid unintended disturbance to the soil, including plug uplift, erosion, and piping.

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

7.6.2.2 Skirt penetration resistance

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

8 Pile foundation design

8.1 Pile capacity for axial compression

8.1.1 General

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

The axial pile capacity shall satisfy the following conditions:

$$P_{\mathrm{d,e}} \leq Q_{\mathrm{d}} = Q_{\mathrm{r}}/\gamma_{\mathrm{R,Pe}} \tag{18}$$

$$P_{\mathrm{d,p}} \leq Q_{\mathrm{d}} = Q_{\mathrm{r}}/\gamma_{\mathrm{R,Pp}} \tag{19}$$

where

Q_{d} is the design axial pile capacity, i.e. the design resistance of the pile;

Q_{r} is the representative value of the axial pile capacity, as determined in 8.1 and 8.2;

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

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

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

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

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

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

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

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

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

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

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

8.1.2 Axial pile capacity

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

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

where

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

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

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

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

A_s is the side surface area of the pile in soil (m^2);

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

A_{pile} is the gross end area of the pile (m^2);

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

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

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

The foundation configurations should be based on those that experience has shown can be installed consistently and practically under similar conditions with the pile size and installation equipment being used. Possible remedial action in the event that design objectives cannot be obtained during installation should be investigated and defined prior to construction.

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

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

8.1.3 Skin friction and end bearing in cohesive soils

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

$$f(z) = \alpha\, s_u(z) \tag{21}$$

where

α is the dimensionless skin friction factor, for clays

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

The factor α can be computed by:

$$\alpha = 0{,}5\ \Psi^{-0{,}5} \text{ for } \Psi \le 1{,}0 \tag{22a}$$

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

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

where

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

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

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

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

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

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

$$q = 9\, s_u \tag{24}$$

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

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

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

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

— the pile achieves penetration of two to three pile diameters or more into the layer in question, and

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

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

8.1.4 Skin friction and end bearing in cohesionless soils

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

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

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

where

β is the dimensionless skin friction factor, for sands;

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

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

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

$$q = N_q \, \sigma'_{vo,tip} \tag{26}$$

where

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

N_q is the dimensionless bearing capacity factor.

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

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

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

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

— the pile achieves penetration of two to three pile diameters or more into the cohesionless layer, and

— the tip is at least three pile diameters above the bottom of the layer to preclude punch-through.

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

Table 1 — Design parameters for cohesionless siliceous soil

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

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

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

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

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

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

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

8.1.5 Skin friction and end bearing of grouted piles in rock

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

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

Design values for (static) unit skin friction and end bearing can be found in various publications.[A.8-13][A.8-20][A.8-21] It is noted that 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, consideration shall be given to the fact that cyclic actions can adversely affect the axial capacity of such piles.

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 cohesive soils, $f(z)$ shall be the same as stated in 8.1.3. For cohesionless soils, $f(z)$ shall be computed in accordance with 8.1.4. For rock, $f(z)$ shall be the same as stated in 8.1.5.

8.3 Axial pile performance

8.3.1 Static axial behaviour of piles

Pile axial deflections should be within acceptable serviceability limits and these deflections shall be compatible with the internal forces and movements of the structure. 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 cohesive 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 capacity of the pile can be less than that given by Formula (20). If t–z curves that exhibit strain-softening are recommended as per 8.4, then maximum axial capacities that explicitly account for the axial flexibility of the pile shall be provided. In these cases an explicit consideration of these effects on ultimate axial capacity is warranted. Note that other factors such as increased axial capacity under loading rates associated with storm waves can counteract the above effects. For more information, see the commentary in ISO 19902, as well as A.8.3.2 and Reference [A.8-2].

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/or an accumulation of deformation. Rapidly applied actions can cause an increase in resistance and/or stiffness of the pile. Very slowly applied actions can cause a decrease in resistance and/or stiffness of the pile. The resultant influence of cyclic actions will be 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 compression

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.

Theoretical curves described in Reference [A.8-3] can be constructed. Empirical t–z curves based on the results of model- and full-scale pile loading tests can follow the procedures described in Reference [A.8-4] for clay soils or described in Reference [A.8-5] for granular soils. Additional curves for clays and sands are provided in Reference [A.8-6]. Resistance–displacement relationships for grouted piles are discussed in Reference [A.8-7].

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 3 are recommended for non-carbonate soils.

A typical value for z_{peak} of 1 % of the pile outer diameter (i.e. $z_{peak}/D = 0{,}01$) is recommended for routine design purposes. However there is significant uncertainty on this value and values ranging from 0,25 % to 2,0 % of the pile diameter can be considered in cases where axial pile stiffness is critical for design.

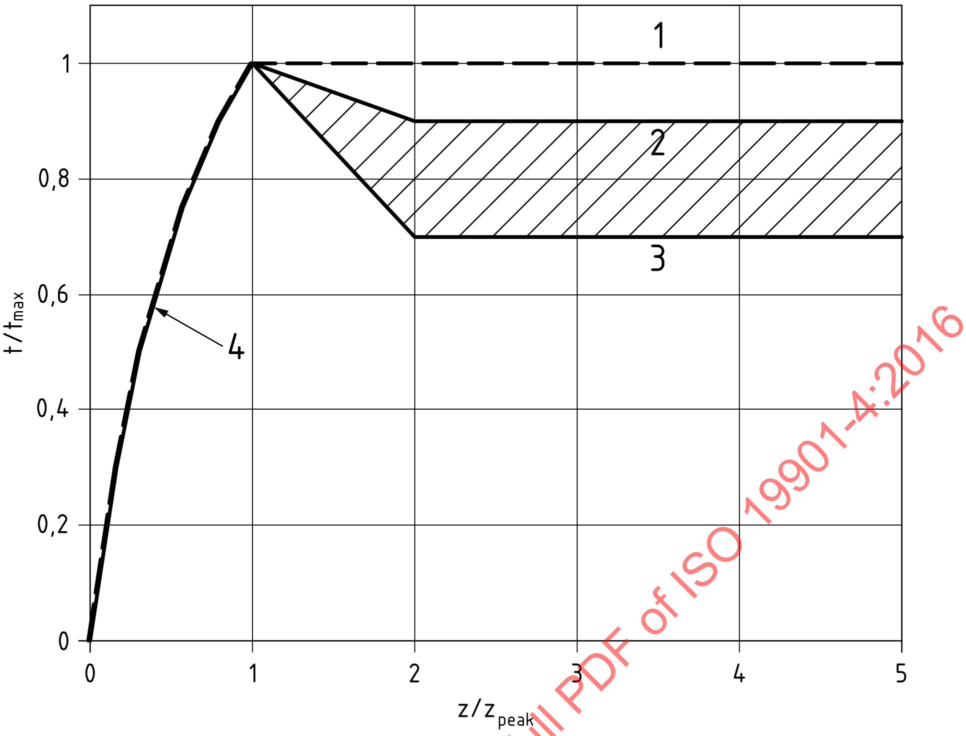
The shape of the t–z curve at displacements greater than that at which t_{max} is reached as shown in Figure 3 should be carefully considered. Values of the residual friction ratio, t_{res}/t_{max}, and the axial pile displacement, z_{res}, at which it occurs, are a function of soil stress–strain behaviour, 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 but laboratory, *in situ* or model pile tests can provide valuable information for determining values of t_{res}/t_{max} and z_{res} for various soils.

For long piles, which are axial flexible, and for soils with strain softening characteristics, the unit skin friction with axial displacement will likely degrade to values lower than those derived using Formulae (21) to (23). Under such conditions, the ultimate axial capacity will be less than that calculated using the methods presented for the representative axial capacity and the degradation shall be accounted for explicitly (see also A.8.1.3.2.4 and A.8.4.1).

8.4.2 End bearing resistance–displacement, Q–z, curve

The relationship between mobilized end bearing resistance and axial pile tip displacement is described using a Q–z curve.

The representative end bearing capacity should be determined as described in 8.1. However, relatively large pile tip displacements are required to mobilize the full end bearing resistance. A pile tip displacement of 10 % of the pile diameter can be required for full mobilization in both sand and clay soils. In the absence of more definitive criteria, the curve shown in Figure 4 is recommended for both sands and clays.



z / z_{peak}	t / t_{max}	
	Clays	Sands
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

1 sand: t_{max}

2 clay: t_{res} = 0,9 t_{max}

3 clay: t_{res} = 0,7 t_{max}

4 clay and sand

z local pile axial displacement

z_{peak} displacement to maximum soil-pile unit skin friction

D pile outside diameter

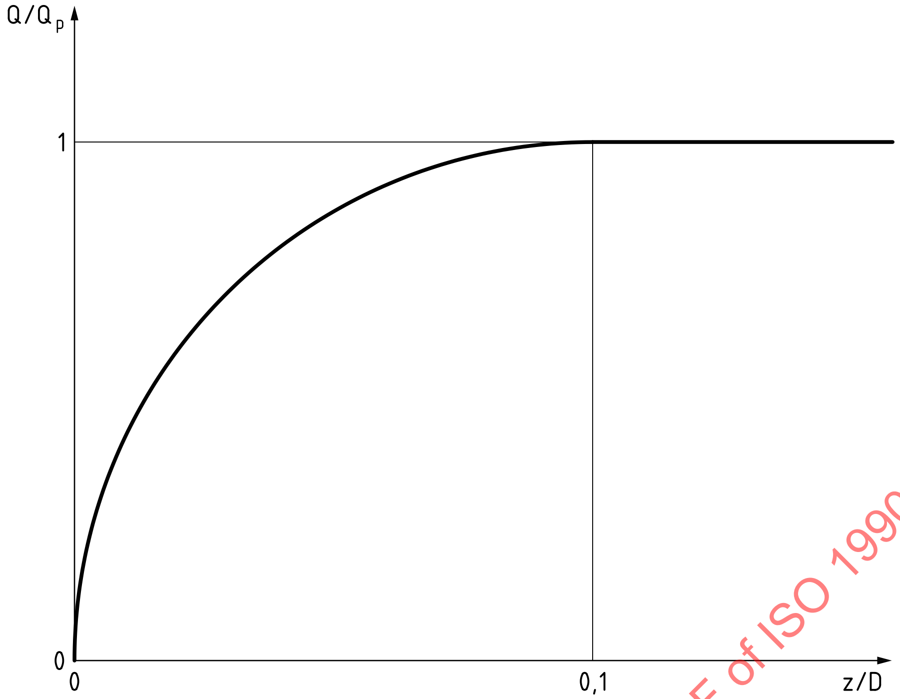
t mobilized soil-pile unit skin friction (in stress units)

t_{max} = $f(z)$ = maximum soil-pile unit skin friction computed in accordance with 8.1 (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 3 — Typical axial pile shear transfer–displacement *t*–*z* curves



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

z axial pile tip displacement

D pile outside diameter

Q mobilized end bearing resistance (in force units)

Q_p representative value of end bearing resistance computed in accordance with 8.1 (in force units)

Figure 4 — Typical pile end bearing resistance–displacement Q–z curve

8.5 Soil reaction for piles under lateral actions

8.5.1 General

The pile foundation shall be designed to resist static and cyclic lateral actions.

The lateral resistance of the soil near the surface is significant to pile design, and the possible effects of scour on this resistance shall be considered. In the absence of more definitive criteria, the procedures given in 8.5.2 to 8.5.7 can be used for constructing the relationships between lateral soil resistance and lateral displacement, p–y curves. Extrapolation of the presented relationships to pile geometries outside the limits of experience should be done with caution. In particular, large diameter piles with limited penetration could require a different formulation for the p–y relationships.

8.5.2 Lateral capacity for soft clay

For static lateral actions, the representative unit lateral capacity, $p_r D$, of soft clay ($s_u \leq 100$ kPa or 2000 lb/ft^2), in units of force per unit length of pile, has been found to vary between $8 \cdot s_u D$ and $12 \cdot s_u \cdot D$ except at shallow depths where failure occurs in a different mode due to low overburden stress. Cyclic actions cause deterioration of lateral capacity below that for static actions. In absence of more definitive criteria, the following value of lateral capacity shall be used

$p_r D$ increases from $3 \cdot s_u D$ to $9 \cdot s_u D$ as z increases from 0 to z_R according to Formula (27):

$$p_r D = 3 \cdot s_u \cdot D + \gamma' z D + J s_u z \tag{27}$$

but $p_r D$ is limited by Formula (28):

$$p_r D = 9 \cdot s_u \cdot D \text{ for } z \geq z_R \tag{28}$$

where

- D is the pile outside diameter;
- p_r is the representative lateral capacity (in stress units);
- s_u is the characteristic value of undrained shear strength at the point in question (in stress units);
- γ' is the submerged soil unit weight (kN/m^3);
- J is a dimensionless empirical constant with values ranging from 0,25 to 0,5 having been determined by field testing;

  NOTE A value of 0,5 is appropriate for Gulf of Mexico clays if no other information is available.
- z is the depth below original seafloor;
- z_R is the depth below soil surface to bottom of reduced resistance zone.

For a condition of constant strength with depth, Formula (27) and Formula (28) are solved simultaneously to give:

$$z_R = \frac{6D}{\dfrac{\gamma' D}{s_u} + J} \tag{29}$$

For non-uniform soils, Formula (27) and Formula (28) can be solved by plotting the two formulae for $p_r D$ versus depth. The point of first intersection of the two formulae is taken to be z_R. In general, z_R is in excess of 2,5 pile diameters. These empirical relationships do not necessarily apply where strength variations are erratic. These formulae also do not apply in case of scour but scour is generally not a concern for cohesive soils.

8.5.3 Lateral soil resistance–displacement *p*–*y* curves for soft clay

Lateral soil resistance–displacement relationships for piles in soft clay are generally nonlinear. The *p*–*y* curves for short-term static actions can be generated from the first column in Table 2. For the case where equilibrium has been reached under cyclic actions, the *p*–*y* curves can be generated from Table 3.

8.5.4 Lateral capacity for stiff clay

For static lateral actions, the representative unit lateral capacity (force per unit length of pile, p_r) of stiff clay ($s_u > 100$ kPa or 2000 lb/ft^2) is similar to that for soft clay. However, due to rapid deterioration under cyclic actions, the representative lateral capacity shall be reduced for cyclic design considerations, in accordance with acceptable best practice or available data.

8.5.5 Lateral soil resistance–displacement *p–y* curves for stiff clay

While stiff clays also have nonlinear stress–strain relationships, they are generally more brittle than soft clays. In developing stress–strain curves and subsequent *p–y* curves for cyclic actions, consideration should be given to the possible rapid deterioration of lateral capacity at large displacements for stiff clays, in accordance with acceptable best practice or available data.

Table 2 — Mobilized lateral resistance — Displacement data for short-term static actions for soft clay

p / p_r	y / y_c
0	0
0,23	0,1
0,33	0,3
0,50	1,0
0,72	3,0
1,00	8,0
1,00	∞

Key

P_r is the representative lateral capacity (in stress units);

p is the mobilized lateral resistance (in stress units);

y is the local pile lateral displacement;

y_c equals $2{,}5 \times \varepsilon_c \times D$;

D is the outside pile diameter;

ε_c is the strain at one-half the maximum deviator stress in laboratory undrained compression tests of undisturbed soil samples.

Table 3 — Mobilized lateral resistance — Displacement data for equilibrium conditions of cyclic actions for soft clay

$z > z_R$		$z < z_R$	
p / p_r	y / y_c	p / p_r	y / y_c
0	0	0	0
0,23	0,1	0,23	0,1
0,33	0,3	0,33	0,3
0,50	1,0	0,50	1,0
0,72	3,0	0,72	3,0
0,72	∞	$0{,}72\ z / z_R$	15,0
		$0{,}72\ z / z_R$	∞

Key

z is the depth below seafloor;

z_R is the depth below seafloor to bottom of reduced capacity zone for uniform soils (see Formula (29);

p_r is the representative lateral capacity (in stress units;

p is the mobilized lateral resistance (in stress units);

y is the local pile lateral displacement (mm);

y_c equals $2{,}5 \times \varepsilon_c \times D$;

D is the pile outside diameter;

ε_c is the strain at one-half the maximum deviator stress in laboratory undrained compression tests of undisturbed soil samples.

8.5.6 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 (30) to a value at deep depths determined by Formula (31). At a given depth, the formula giving the smallest value of p_r should be used as the representative capacity. These formulae can be un-conservative for layered soil conditions when the sand is overlain by soft clay.

$$p_{rs} = (C_1 z + C_2 D)\, \gamma' z \tag{30}$$

$$p_{rd} = C_3 D\, \gamma' z \tag{31}$$

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);

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

z is the depth below original seafloor (m);

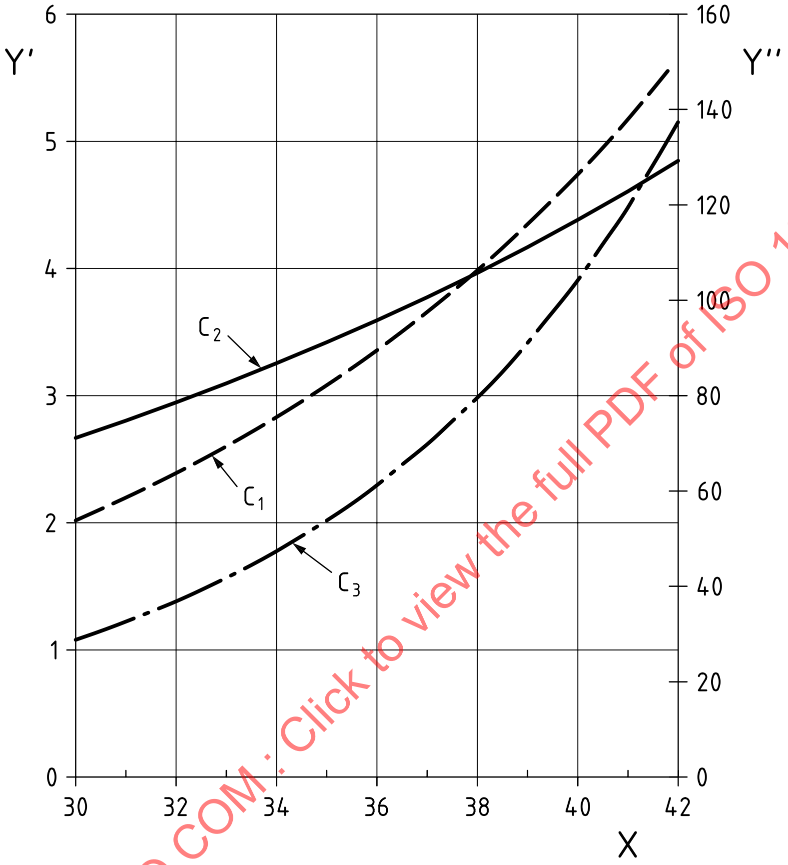
C_1, C_2, C_3 are dimensionless coefficients as a function of the effective angle of internal friction in sand, ϕ' (see Figure 5):

$$C_1 = \frac{(\tan\beta)^2 \tan\alpha}{\tan(\beta - \phi')} + K_o \times \left[\frac{\tan\phi' \times \sin\beta}{\cos\alpha \times \tan(\beta - \phi')} + \tan\beta \times \left(\tan\phi' \times \sin\beta - \tan\alpha \right) \right] \tag{32}$$

$$C_2 = \frac{\tan\beta}{\tan(\beta - \phi')} - K_a \tag{33}$$

$$C_3 = K_a \times \left[(\tan\beta)^8 - 1 \right] + K_o \times \tan\phi' \times \left(\tan\beta \right)^4 \tag{34}$$

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



Key

X effective angle of internal friction (ϕ' in degrees)
Y' value of coefficients C1 and C2
Y'' value of coefficient C3
C_1, C_2, C_3 coefficients for lateral capacity

Figure 5 — Lateral capacity coefficients for sand

Guidance for scour conditions is given in A.8.5.6.

8.5.7 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 definitive information can be approximated at any specific depth, *z*, by:

$$p = A \cdot p_r \cdot \tanh\left(\frac{k \cdot z}{A \cdot p_r} \cdot y \right) \tag{35}$$

where

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

$$A = \left(3,0 - \frac{0,8 \cdot z}{D}\right) > 0,9 \qquad \text{for static actions, and} \tag{36}$$

$A = 0,9$ 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 4;

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°, should be done with caution. In particular, laboratory test results on such soils should be critically reviewed for evidence of anomalous behaviour and for the presence of significant fractions of cohesive soils, either of which could require a different formulation for the p–y relationships.

In the absence of more definitive information, the values of the initial modulus of subgrade reaction, k, given in Table 4 are recommended.

Table 4 — Initial modulus of subgrade reaction

ϕ'	k	
	MN/m^3	(lb/in^3)
25°	5,4	(20)
30°	8,7	(32)
35°	22	(80)
40°	45	(165)

8.6 Pile group behaviour

8.6.1 General

Consideration should be given to the effects of closely spaced adjacent piles on the resistance–displacement characteristics of the pile group. Generally, for pile spacing less than eight diameters, group effects should be evaluated.

The pile group 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 in the group can be less than those specified in 8.1.1, provided it can be 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 cohesive or cohesionless soils, 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 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 [A.8-8], the following methods were found to be the most appropriate for use in designing group pile foundations for the given loading conditions:

— for defining initial group stiffness: Advanced methods, such as PILGP2R;[A.8-8]

— for design event actions: the Focht-Koch method[A.8-9] as modified by Reese et al.[A.8-10] 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;

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

9 Pile installation assessment

9.1 General

The types of pile foundations used to support offshore structures and considered in this part of ISO 19901 are:

— Driven piles: Open-ended piles are normally used in foundations for offshore structures (see 9.2 to 9.7). These piles are usually driven into the seabed with impact hammers, which use steam, diesel fuel or hydraulic power as the source of energy;

— Drilled and grouted piles, which can be used in soils and rocks which will hold an open hole with or without drilling mud (see 9.8);

— Belled piles: Bells can be constructed at the tip of piles so as to give increased bearing and uplift capacity through direct bearing on the soil (see 9.9). The end bearing capacity of belled piles shall be determined in accordance with the principles given for the design of drilled and grouted piles;

— 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 cohesionless soils. Owing to the lack of data with respect to the effect of the installation method on the pile axial capacity, the use of vibratory hammers for installing offshore piles subjected to significant axial actions is not recommended.

The pile wall thickness shall be adequate 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.

Proper installation of piles, including conductor piles, is vital to the life and permanence of the structure and requires each pile to be installed to or near design penetration without damage. All field-made structural connections shall be compatible with the design requirements. Pile sections should be marked in a manner to facilitate installing the pile sections in the proper 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.

9.2 Drivability studies

Computer analyses (based on the principles of one-dimensional elastic stress-wave theories and commonly known as wave formula analyses) can 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), but will also be affected 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 formula model. Therefore, selection of these input parameters should be based on previous pile driving experience and engineering judgment. For a given rated energy, the energy transferred to the pile is highly dependent on the type of hammer (i.e. diesel fuel, steam, or hydraulic hammer) and should be based on pile driving experience with reliable measurements from pile instrumentation.

The definition of SRD is the main factor that governs the results of the drivability studies, i.e. 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.9.2).

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

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

Pile driving instrumentation data, based on measurements from strain and acceleration transducers fixed near the top of the pile, can 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 actual SRD 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.

9.3 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. It is often necessary to work one pile at a time during the driving of the final one or two sections, so as to minimize set-up time. 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 ensure that it is capable of supporting the design actions. Final blow count alone cannot be considered as assurance of piling adequacy. Continued driving beyond the defined refusal (see 9.4 and A.9.4) can be justified if it offers a reasonable chance of significantly improving the capacity of the foundation without risk of damaging the pile, hammer or structure.

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 9.5 (to be agreed with the design engineer).

9.4 Driven pile refusal

Pile refusal requires defining, primarily in order to

— establish the point at which pile driving with a particular hammer should be stopped and other methods instituted (see 9.5), and

— 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 in the event that no other requirements are included in the installation specification, are given in A.9.4.

If a pile refuses before it reaches design penetration, one or more of the measures given in 9.5 can be taken.

9.5 Pile refusal remedial measures

9.5.1 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.

9.5.2 Re-evaluation of design penetration

Reconsideration of actions, displacements and required capacities of individual piles, of other foundation elements and of the foundation as a whole, 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 increased.

9.5.3 Modifications to piling procedures

9.5.3.1 General

Modifying procedures, usually the last course of action, can permit the piles to be driven to the required penetration. The modifications described in 9.5.3.2 to 9.5.3.4 can be considered.

9.5.3.2 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 ensure that sufficient pile capacity is regained. The minimum axial capacity of the plug shall be equal to the pile end bearing capacity in a plugged condition. Attention shall be paid to the characteristics of shear transfer between plug and pile. In some circumstances plug removal is not effective, particularly in cohesive soils.

9.5.3.3 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 considered 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, in general, be avoided because of the unpredictability of the results.

9.5.3.4 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.

9.6 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 should be used to determine the dynamic stresses generated by hammer impact.

The type(s) of pile hammer considered for pile driving shall be noted by the designer on the installation drawings or specifications. Any change in the hammers to be used for pile driving shall be assessed, in order to ensure 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 more particularly relevant to pile design and installation assessment are:

— Stresses during driving: The unfactored dynamic stresses should not exceed 80 % to 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.

— Allowance for underdrive or overdrive: With piles having thickened sections at the seafloor, consideration shall be given to providing an extra length of heavy wall material in the vicinity of the seafloor 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.

— 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 should 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 should 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.

9.7 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 through sleeves alone without lateral restraint. In calculating pile stresses, full account should be taken of wave, current and wind actions, 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 by the installation contractor to maintain a fairly 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. It is therefore important that measurements be made to give a record of the hammer's performance, including ram impact velocity, stroke, pressure of accelerating medium, and blow rate. Reliable instrumentation of some piles should 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, it can be considered to extend the hammer casing or to use followers 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. However, because of the steel-to-steel contact, particular attention should be paid to the design of the pile head.

In selecting hydraulic hammers for deeper 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. It should be noted that hydraulic hammer changes can take longer time than steam hammer changes.

9.8 Drilled and grouted piles

There are two types of drilled and grouted piles.

— 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.

— 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, consideration should be given to flushing the mud 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 be done carefully, in order to 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 can 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 it can be necessary to tie the pile 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 assurance that this will not be detrimental to pile capacity and that grout will not migrate during placement to an adjacent hole.

9.9 Belled piles

Drilling of the bell is carried out through the pile by under-reaming with an expander tool. A pilot hole can be drilled below the bell to act as a sump for unrecoverable cuttings. The bell and the pile are filled with concrete to a height sufficient to develop the necessary transfer of forces between the bell and the pile. Bells are connected to the pile to transfer both full uplift and compressive forces using steel reinforcing such as structural members with adequate shear lugs, deformed reinforcement bars or prestressed tendons.

9.10 Grouting pile-to-sleeve connections

The grout-to-steel bond in the connection between pile and sleeve shall be checked in accordance with ISO 19902:2007, 15.1, addressing grouted connections.

9.11 Pile installation data

Throughout the driving of main or skirt piles, comprehensive driving and associated data should be recorded and reviewed for conformance with the installation plan. If significant deviations are observed, it can be necessary to take appropriate measures. The recorded data can include:

— 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/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);

— 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 or belled piles.

9.12 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 disturbances 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.

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. It should be noted that these detrimental effects can occur whether the drilling takes place either after installation of the structure or before, e.g. for a pre-installed template or for an exploration well. The proximity of conductor slots to as-installed or future pile locations is critical and risks are clearly greater for narrow structures with vertical piles.

The following three recommendations should be considered 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.

— Particular care should be exercised in the installation of conductors which are installed by drilling or drill-drive techniques instead of by driving alone.

— 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 to the structure's design engineer. 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 should be assessed. The cuttings from the well drilling operation, if allowed to accumulate on the seafloor, should be taken into account in the foundation design, installation procedure and structure removal.

The skin friction capacity of conductors installed in cohesive soils by jetting is not covered by the methods described in 8.1.3 and A.8.1.3. Additional guidance is provided in A.9.12.

10 Soil-structure interaction for auxiliary subsea structures, risers and flowlines

Geotechnical considerations for the soil-structure interaction for auxiliary subsea structures, risers and flowlines are given in A.10.

11 Design of anchors for floating structures

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

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 subclause in the body of this part of ISO 19901.

A.1 Scope

There is a large body of technical literature on offshore geoscience studies and foundation design. There are also regular conferences on these topics. The most up-to-date publications can generally be found in:

— the proceedings of the Offshore Technology Conference (OTC);

— the proceedings of the international conferences on the Behaviour of Off-Shore Structures (BOSS), on Offshore and Polar Engineering (ISOPE), and on Offshore Mechanics and Arctic Engineering (OMAE);

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

— the proceedings of the international symposium on Frontiers in Offshore Geotechnics (ISFOG).

General guidance on the application of soil mechanics theory to foundation design can be found in various undergraduate and post-graduate textbooks.

An example of foundation caissons applied to a jacket platform is given in Reference [A.1-1]. Guidance on the design of intermediate foundations with $1 < L/D < 10$ can be found in the literature, for example, Reference [A.1-2].

A.2 Normative references

No additional guidance is offered.

A.3 Terms and definitions

No additional guidance is offered.

A.4 Symbols and abbreviated terms

No additional guidance is offered.

A.5 General requirements

A.5.1 General

No additional guidance is offered.

A.5.2 Design cases and safety factors

No additional guidance is offered.

A.5.3 Characteristic values of soil properties

Characteristic values of soil properties should be estimated for each soil stratum. Soil stratification for a calculation model can differ from the actual stratification of the as-found soil.

Particular caution should be exercised in the utilization of a strength value that depends 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.

Multiple sets of characteristic values for soil properties can be required for a single calculation model, for example because of assumptions made in the calculation model regarding direction, inclination or point of application of actions.

A problem can be such that it stretches a calculation model at its limit of intended applicability. For this situation, the uncertainties should be assigned at their source, i.e. as model uncertainty or as uncertainty of soil properties or both.

Consideration should be given to a priori knowledge. Examples of a priori knowledge include (i) a thin, continuous failure surface that can be inferred from geological conditions, (ii) a known location of a geological fault, (iii) a lower value of angle of internal friction corresponding to constant volume conditions during shear, and (iv) a likely low value for residual strength under undrained conditions.

The characteristic value of a soil property for a soil stratum should be estimated so that it is representative of the actual volume of soil to be considered in the calculation to be done. The actual volume of soil is the domain influencing the occurrence of the limit state under consideration, for example a potential failure zone or failure surface. The domain of influence is also a function of the nature (i.e. magnitude, location and geometry) of the actions and of the characteristics of the soil heterogeneity under consideration.

When selecting a characteristic value, two factors should be considered, (i) the spatial averaging of properties over a potential failure surface which can reduce the uncertainty of a property value, and (ii) the tendency for a failure surface to follow the path of least resistance which can cause an apparent reduction in the property mean.

The use of statistical methods should allow reducing subjectivity and quantifying uncertainties. Statistical approaches form a rational tool for handling uncertainties in soil data, both in terms of natural variability and of limited amounts of data.[A.5-1][A.5-2][A.5-3][A.5-4] The usefulness of statistical methods for the analysis and representation of soil data depends on the quantity and quality of data available. Specialist advice should be sought by the designer when relevant.

A.5.4 Testing and instrumentation

No additional guidance is offered.

A.6 Geotechnical data acquisition and identification of hazards

A.6.1 General

Shallow seismic survey data can allow geotechnical data to be extrapolated from soil borings. However, the degree of correlation between the geophysical and geotechnical data or the distance over which the stratigraphy from a borehole using geophysical results can be extrapolated depends on the nature and quality of the data, as well as on the geology of the site and the soil characteristics themselves.

Further information on using the results of integrated geoscience studies to perform a geotechnical assessment can be found in References [A.6-1] and [A.6-2].

A.6.2 Shallow geophysical investigation

No additional guidance is offered.

A.6.3 Geological modelling and identification of hazards

A.6.3.1 General

No additional guidance is offered.

A.6.3.2 Earthquakes

Additional guidance is provided in ISO 19901-2.

A.6.3.3 Fault planes

This guidance addresses special considerations associated with the presence of non-seismogenic faults at the site of a platform, and how to account for their presence in foundation pile design.

A.6.3.3.1 Fault identification and mapping

Soil borings, soil sampling, and *in situ* testing (e.g. cone penetrometer) typically are not effective in detecting the presence of faults in predominantly clay strata, such as are encountered in the Gulf of Mexico and in deep waters worldwide.

The most effective way to identify and define the extent of faults is through the use of a high-resolution geophysical survey designed to define the details of the foundation-zone stratigraphy. Examples of such surveys include sub-bottom profiler surveys [for example, an Autonomous Underwater Vehicle (AUV) survey in deepwater] or other single- or multi-channel, ultra high-resolution 2-D or 3-D seismic surveys. Whenever feasible, a pair of survey cross-lines should be run directly over each proposed foundation location.

If faults are identified within the foundation zone, their actual or projected intersection with the seafloor should be accurately mapped in relation to the proposed foundation locations on a seafloor topographic map or suitable seafloor image.

A.6.3.3.2 Relative locations of faults and foundations

When the seafloor trace of a fault (or projected trace of any fault in the foundation zone) comes within 150 m (500 ft) of a proposed foundation location, the fault should be defined in 3-D space based on its dip angle and direction and trace azimuth. At a minimum, the fault should be indicated on the seismic cross-section(s) closest to the foundation, and the projection of the foundation should also be indicated on the seismic cross-section. The foundation should be positioned on the cross-section to clearly demonstrate the relative locations of the fault and the foundation, taking into consideration foundation offset from the seismic cross-section, geometric relationships between the fault plane and the seismic cross-section, and seismic velocity, as examples.

A.6.3.3.3 Design considerations

A.6.3.3.3.1 Exclusion zones

It is always good engineering practice to avoid geohazards, whenever possible. If the foundation location can be moved easily so as to avoid the proximity of a fault, this should be the preferred solution. Reference [A.6-3] describes an approach to define an ‘exclusion zone’ around each fault, in which a foundation should not be located. This design approach is that of geohazard avoidance, as further described in Reference [A.6-3] and consists of several steps:

— map the fault trace(s) or upward projection of the fault plane(s) on a seafloor map;

— quantify the uncertainty in the fault plane location due to survey positioning uncertainties, geological interpretation uncertainty, and fault-dip angle uncertainty, and then define the zone in 3-D space (both in plain view and vertical cross-section below the seafloor) in which the fault plane could be located (the ‘fault-plane exclusion zone’). When defining the exclusion zone, the potential for possible future lateral extension (growth) of the fault plane along trend for faults that currently terminate in the area of interest should also be considered;

— locate the foundation a safe distance outside the exclusion zone.

A.6.3.3.3.2 Fault-foundation intersections

In cases when the foundation location cannot be changed easily, it becomes necessary to investigate and quantify the effect(s) the presence of a nearby fault might have on the performance of the foundation, if any.

The foundation systems for several floating platforms for which some of the anchors were located near faults, or where the anchor/foundation piles were expected to intersect a fault plane, have been successfully designed and installed in the Gulf of Mexico, as documented in References [A.6-3] to [A.6-5].

A possible approach consists of several steps.

— Map the fault trace(s) or upward projection of the fault plane(s) on a seafloor map and quantify the uncertainty in the fault plane location, as per A.6.3.3.3.1.

— Determine the minimum offset range between the ‘fault-plane zone’ and the foundation.

— Determine if the fault is still moving (or ‘active’, but not seismically) and if it is, the expected rate of movement.

— If the foundation intersects the fault plane, perform the necessary soil–foundation interaction analyses (e.g. beam-column, finite element) to evaluate the effects (if any) the maximum amount of fault movement expected to occur during the life of the platform could have on the foundation’s integrity and performance.

A.6.3.3.3.3 Effects on soil properties

The design of foundations in the immediate vicinity of faults should also consider the potential effects a fault might have on the soil characteristics along the fault plane and in its direct vicinity.

There is general evidence that some (but not all) faults or some points along some faults are clearly preferential and, sometimes, major fluid migration pathways. In general, the larger (deeper) the fault, the higher the potential that fluid migration can occur or has occurred along the fault. The potential for fluid migration along fault planes, and the potential effects thereof, need to be considered on a case-by-case basis, with heavy reliance on geophysical and geological evidence.

Designers should consider the possible zone of disturbance around fault planes and the effect, if any, it might have on soil properties. However, unless proven otherwise by *in situ* tests or the testing of samples retrieved at or adjacent to the fault plane, there is no strong evidence that an inactive or a very slow moving compaction (or growth) fault would have a deleterious effect on the soil properties of the surrounding soils. Therefore, in general, no reduction in shear strength is recommended to evaluate the anchor/foundation capacity and behaviour.

A.6.3.4 Seafloor instability

No additional guidance is offered.

A.6.3.5 Scour and sediment mobility

No additional guidance is offered.

A.6.3.6 Shallow gas

No additional guidance is offered.

A.6.3.7 Seabed subsidence

Guidance on how to assess the magnitude of possible seabed subsidence can be found in Reference [A.6-6].

A.6.4 Carbonate soils

A.6.4.1 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 northwestern Australian shelf.

The comments in A.6.4.2 to A.6.4.5 are focused 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.6.4.2 Characteristic features 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 symposium on Performance and Behaviour of Calcareous Soils, sponsored by ASTM committee D-18 on soil and rock, Ft. Lauderdale (1981);

— the proceedings of the specialized international conferences on Engineering of Calcareous Sediments, held in Perth (1988) and in Bahrain (1998);

— References [A.6-7] to [A.6-35].

A.6.4.3 Properties of carbonate soils

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:

— material composition, particularly carbonate content;

— material origin to differentiate between skeletal and non-skeletal sediments;

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

— compressibility of the material;

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

— formation cementation, at least in a qualitative sense.

For site characterization, maximum use of local experience is important, particularly in the selection of an appropriate soil investigation and testing programme. In new unexplored territories, 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.

No universally recognized classification system is presently available for carbonate materials. Classification charts for carbonate soils and rocks have been tentatively developed,[A.6-11][A.6-30] 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.

A.6.4.4 Foundations in carbonate soils

A.6.4.4.1 Driven piles

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

Piles installed by driving in carbonate soils can free-fall at stab-in, under hammer weight or during the driving process. The possibility of pile free-fall should be assessed. The use of pile arrestor or other method to reduce the speed of free-fall or to stop the pile should be considered where appropriate.

On the other hand, dense, strongly cemented carbonate deposits can be very competent foundation material. Unfortunately, the difficulty in obtaining high-quality samples and the lack of generalized design methods sometimes makes it difficult to predict where problems can occur. With clays, care should be taken when the carbonate content exceeds 50 % and where no pile test data or local experience exists.

A.6.4.4.2 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 mobilize 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.[A.6-10] 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, consideration should be given to the potential for reduction in friction capacity due to cyclic actions, especially once slip has been initiated between pile and soil.

A.6.4.4.3 Shallow foundations

Shallow foundations are suitable for use on carbonate sediments, although any evaluation of such foundations should account for the important differences that exist with such material compared with silica sands or normal clays. 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. It should also be noted that 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.

Shallow foundations are 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 should be treated cautiously, with due account taken of the risks and consequences of a punch-through type of failure. Foundation systems using suction assistance for penetration of skirts should be evaluated carefully on a case-by-case basis.

A.6.4.5 Assessment

To date, general design procedures for foundations in carbonate soils are not available. Acceptable design methods have evolved but remain highly site-specific and dependent on local experience. Stemming from some recent 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 Design of shallow foundations

A.7.1 General

The formulae provided in this subclause are limited in nature and are not necessarily appropriate for design in a number of situations. A common situation for which they cannot be applied is if a strong layer overlies a weak layer within the zone of influence of the foundation. Irregularly shaped foundations are also difficult to analyse using the formulae provided.

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

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, subject to appropriate documentation and justification.

A.7.2 Principles

A.7.2.1 General principles

No additional guidance is offered.

A.7.2.2 Sign conventions, nomenclature and action reference point

No additional guidance is offered.

A.7.2.3 Action transfer

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

— soil resistance on the sides of the embedded foundation,

— submerged foundation weight, and

— submerged soil weight within skirts (if applicable).

Partial action factors should be applied to the foundation weight and soil weight. The factor for soil weight is generally equal to unity, however higher or lower factors can be considered in some cases, especially where there is uncertainty and the soil weight leads to improved foundation stability.

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

— horizontal passive and active soil resistance, and

— 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 and specialist advice should be sought.

A.7.2.4 Idealization of foundation area and the effective area method

A.7.2.4.1 Idealization of foundation area

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:

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

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

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

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

A.7.2.4.2 Effective area method

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.1 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. Q and M should include appropriate partial action factors, being mindful that increasing the vertical action up to a value of $0{,}5Q_{\text{ult}}$ will increase moment capacity. The partial 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 the skirt compartment encapsulates soil of sufficiently low permeability, the vertical action used to calculate the effective area can include a contribution due to soil trapped within the skirted area. The following is noted.

— 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).

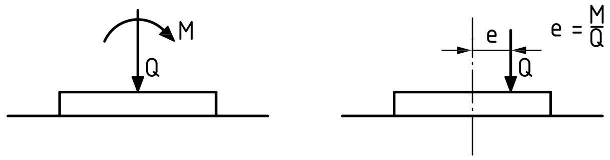
— Where it is considered appropriate to use the weight of the soil plug, the submerged soil weight should be used. Specialist geotechnical advice should be sought to ensure inclusion of the soil plug does not lead to less conservative foundation design. The latter comment relates specifically to soft soil sites, where the design soil strength is insufficient to support the submerged soil weight at the depth of the skirts, sufficient support is an implicit assumption in the use of an effective area approach. In some cases, it might be necessary to adopt an alternative approach to account for moment loading (such as the yield surface method, see A.7.3.5).

— The submerged unit weight used in the analysis should be based on site investigation and laboratory data, and should incorporate an appropriate level of conservatism to account for any uncertainty. Generally, it is conservative to adopt a lower bound profile of submerged soil weight.

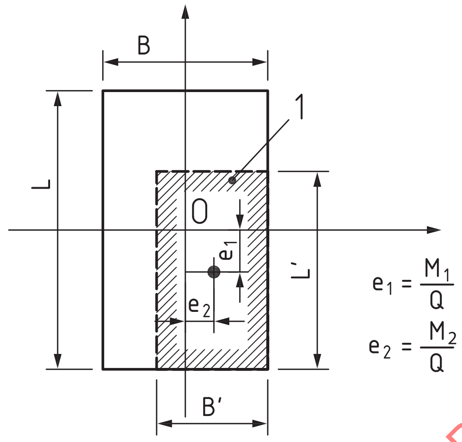
For a rectangular base area [Figure A.1 b)], eccentricity can occur with respect to either axis of the foundation. Ideally, in conditions where eccentricity occurs in two directions specialist geotechnical advice should be sought. A simplified means of addressing this is to reduce the dimensions of the foundation in both directions:

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

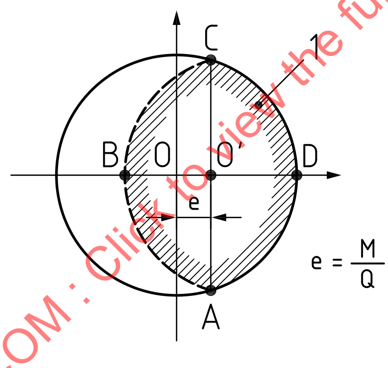
With $B' \leq L'$ and where L and B are the foundation length and width, respectively, the prime denotes effective dimensions, and e_1 and e_2 are eccentricities along the length and width.



a) Equivalent loadings



b) Reduced area — Rectangular foundation



c) Reduced area — Circular foundation

Key

1 reduced area

Figure A.1 — Definition of effective area for various foundation geometry

Circular foundations subject to eccentric actions can be idealized as rectangular foundations by solving Formulae (A.1) to (A.3). Alternatively, for a circular base with radius, R, the effective area can be assumed as shown in Figure A.1 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:

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

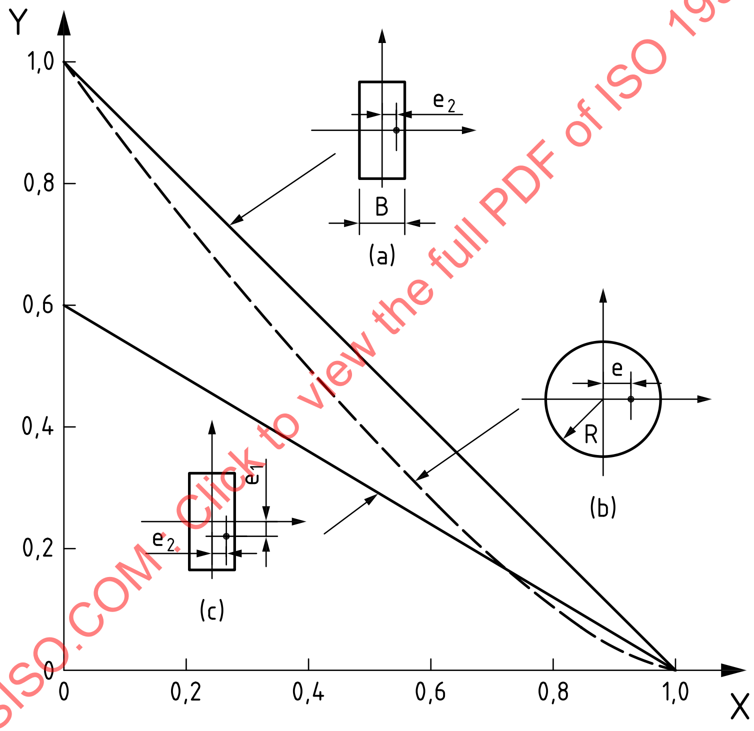
$$L' = \left(2s\sqrt{\frac{R+e}{R-e}}\right)^{1/2} \qquad \text{(A.5)}$$

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

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.2 in a dimensionless form.



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.2 — Area reduction factors for eccentrically loaded shallow foundations

No published data are available on other foundation shapes. If appropriate, intuitive 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 [A.7-1]. Alternatively, an idealized rectangular foundation can be determined by solving Formula (A.1) to Formula (A.3).

Alternative methods exist for assessing the effect of eccentricity in multiple or non-orthogonal directions[A.7-2] and these can be more applicable in more complex conditions.

A.7.3 Acceptance criteria and design considerations

A.7.3.1 Action and material factors

No additional guidance is offered.

A.7.3.2 Use in design

No additional guidance is offered.

A.7.3.3 Special cases

No additional guidance is offered.

A.7.3.4 Additional design considerations

A.7.3.4.1 Adjusting for soil plug weight in skirted foundations

No additional guidance is offered.

A.7.3.4.2 Adjusting for horizontal seabed resistance above foundation level

Contributions to horizontal resistance of an embedded foundation can come from (i) base shear, (ii) soil resistance above skirt tip level due to the difference between active and passive resistance, and (iii) 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 regards to active and passive resistance, the following advice 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, it can be appropriate to also account for the weight of soil within the passive soil wedge, although in such cases it should be verified that the total lateral resistance calculated does not exceed that which would be calculated using $K_{ru} = 4$.

In some soils it might not be appropriate to include the full soil resistance above skirt tip level in assessing overall sliding stability, due to strain compatibility issues, and specialist advice should be sought in these cases.

b) Drained conditions

The drained horizontal soil reaction factor K_{rd} depends on several factors, such as mobilized soil friction angle, roughness, foundation shape, side shear, depth of embedment, and possible side gap between foundation and soil from installation or from scour. Provided that the installation procedure and/or other foundation aspects do not require a more accurate assessment of the drained horizontal soil reaction factor, the following formula is recommended.

$$K_{\mathrm{rd}} = K_{\mathrm{p}} - \left(\frac{1}{K_{\mathrm{p}}}\right) \tag{A.6}$$

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

$$K_{\mathrm{p}} = \left[\tan\left(\frac{\pi}{4} + 0{,}5\arctan\left(\frac{\tan\phi'}{\gamma_{\mathrm{m}}}\right)\right)\right]^2 \tag{A.7}$$

A.7.3.4.3 Shallow foundations penetrating into soft soils

No additional guidance is offered.

A.7.3.4.4 Tensile stresses beneath foundations

No additional guidance is offered.

A.7.3.4.5 Non-standard 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. For example, the potential of a deep bearing failure depends on soil strengths at considerably greater depths than that of a sliding failure. Hence careful attention should be given to defining the soil parameters throughout the expected zone of influence.

Where foundation conditions are highly heterogeneous or anisotropic; where loading conditions deviate considerably from the simple conditions assumed in the bearing capacity formulae; where loading rates are such that the conditions are not clearly drained or undrained; or where foundation geometries are highly irregular, the use of the standard stability formulae presented in this Annex are not applicable and alternative procedures such as one or combinations of the following should be selected:

— use of conservative equivalent parameters along with the recommended formulae;

— 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 properly scaled model tests to check and verify calculation models and procedures.

A.7.3.4.6 Interaction with other structures

Additional guidance is provided in ISO 19905-1 for interaction with jack-up spudcans and in 9.12 of this part of ISO 19901 for interaction with conductors.

A.7.3.4.7 Multiple foundations

In many instances use of multiple shallow foundations can significantly increase overall foundation capacity, as illustrated in References [A.7-3] and [A.7-4].

A.7.3.4.8 Consideration of surrounding seabed conditions

No additional guidance is offered.

A.7.3.4.9 Carbonate soils

Refer to A.6.4.4.3 for shallow foundations on carbonate soils.

A.7.3.5 Alternative method of design based on yield surfaces

A.7.3.5.1 General

An alternative method of design to assess foundation stability under general loading makes use of yield surfaces, as described in A.7.3.5.2 to A.7.3.5.7. Specialist geotechnical advice should be sought where this method is to be used for offshore design.

A.7.3.5.2 Background

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 traditional design approach for shallow foundations involves transforming the combined action into an equivalent vertical and lateral loading acting on a reduced (effective) foundation area.

Restrictions apply in the use of the traditional design approach for offshore structures (such as in regards to allowing net tensile stress changes in the soil) which limit its general applicability. Recent literature also suggests that predicting bearing capacity of shallow foundations using the effective area method can lead to considerable under-prediction of capacity for some loading situations, as illustrated in References [A.7-5] to [A.7-7].

An alternative approach is to derive a fully encompassing yield surface in Q, H, M and T space. This can be used to predict loading combinations to reach ultimate limit state, as well as an explicit indication of the effect of a change in individual 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.

NOTE The effects of torsion are currently excluded and specific guidance on the impact of torsion loading on derivation of yield envelopes is not provided. The effect of torsion is explicitly considered in References [A.7-23].

A.7.3.5.3 Application to design

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

— 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.

— 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 characteristic value of s_u for undrained conditions and to $\tan\phi'$ for drained conditions (not to ϕ').

A.7.3.5.4 Prediction of uniaxial capacity

Uniaxial capacity Q_{ult}, H_{ult} and M_{ult} for undrained conditions can be derived from existing recommendations, although more accurate and rigorous solutions have become available in recent years. Key references include

— for Q_{ult}, References [A.7-8] to [A.7-13], and

— for H_{ult} and M_{ult}, References [A.7-14] to [A.7-19].

Algebraic expressions are available to describe the shape of yield surface for a selection of cases. A review of the current literature for these cases is discussed in A.7.3.5.5 to A.7.3.5.7.

A.7.3.5.5 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.7.3.5.6 Undrained conditions

A.7.3.5.6.1 Surface foundations with zero-uplift capacity along the foundation–soil interface

The general 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.3 in terms of normalized actions, Q/Q_{ult}, H/H_{ult} and M/M_{ult} (vertical action Q is denoted by V in the figure). 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 seabed.

Reference [A.7-20] presents yield surfaces for rectangular surface foundations and shows the normalized shape of the yield surface is unique for aspect ratios over the range $0 < B/L < 1$, and presents approximating expressions for the shape of the yield surface and the uniaxial capacities defining its apex points. References [A.7-21] presents a similar 'scallop-shaped' yield surface for circular foundation geometry but does not present an approximating expression to enable direct comparison of the shape of the yield surface.

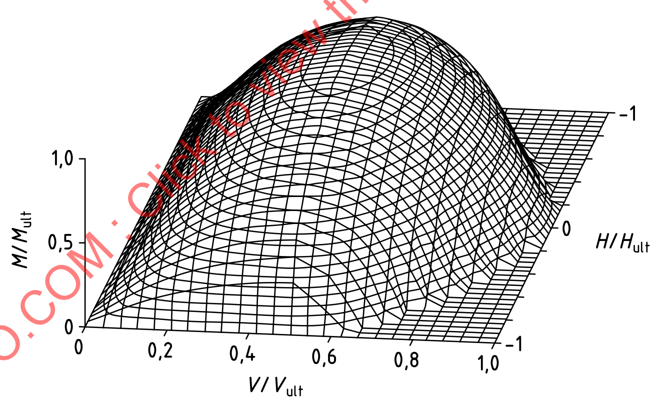


Figure A.3 — Yield surface for undrained conditions for a surface foundation with zero-uplift capacity along the foundation–soil interface[A.7-21]

A.7.3.5.6.2 Surface foundations with unlimited-uplift capacity along the foundation–soil interface

In some cases, uplift resistance can be mobilized beneath surface or skirted foundations due to suction and can potentially be relied on for the duration over which undrained conditions prevail. Engineering judgment is required to determine whether suctions will be generated and the duration over which they can be maintained.

Uplift resistance provided by foundation skirts can be conceptually 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.4 (vertical load Q is denoted by V in the figure). The 'walnut-shaped' surface is asymmetric in the H–M plane, with maximum moment capacity mobilized 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 seabed.

Yield surfaces have been derived for strip, rectangular and circular foundations, and homogeneous and heterogeneous soil strength, although not comprehensively for all combinations. Algebraic expressions have been derived for the selected cases of strip and rectangular foundations and linearly increasing shear strength[A.7-14][A.7-23] and a circular foundation and uniform shear strength.[A.7-22]

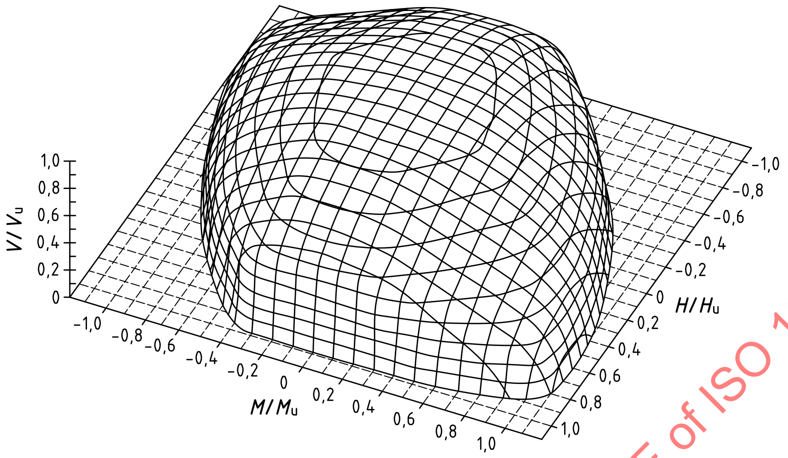


Figure A.4 — Yield surface for undrained conditions for a surface foundation with unlimited-uplift capacity along the foundation-soil interface[A.7-15]

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 would be un-conservative.[A.7-6][A.7-20]

A.7.3.5.6.3 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.5 (vertical action, Q, is denoted by V in the figure).

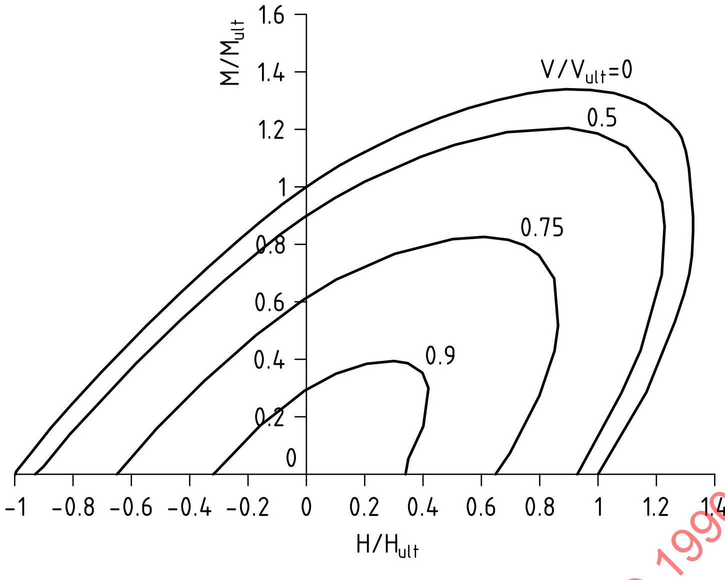


Figure A.5 — Yield surface for undrained conditions for an embedded foundation[A.7-19]

References [A.7-17] to [A.7-19] present yield surfaces in general loading space (Q, H, M) for strip and circular shallow foundations with embedment ratios in the range zero to one for uniform soil strength and linearly increasing strength with depth. Reference [A.7-23] presents uniaxial limit states and a yield surface for surface and shallowly embedded rectangular foundations and linearly increasing soil shear strength profiles for embedment ratios up to 0,2.

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.[A.7-25] 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 [A.7-26] and [A.7-27].

Key references relating to yield surfaces in undrained conditions include References [A.7-5], [A.7-6] and References [A.7-14] to [A.7-25].

A.7.3.5.7 Drained conditions

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 $Q < 0,5\ Q_{ult}$.

A.7.3.5.7.1 Surface foundations

The general form of the yield surface for drained failure of a surface foundation is shown in Figure A.6. 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 the figure). Maximum horizontal action and moment capacity are mobilized in conjunction with a vertical action $Q = 0,5\ Q_{ult}$ and maximum moment capacity is mobilized in conjunction with horizontal action acting in opposition (i.e. clockwise and right-to-left or vice versa).

Reference [A.7-28] proposed the yield surface shown in Figure A.6 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.[A.7-29]

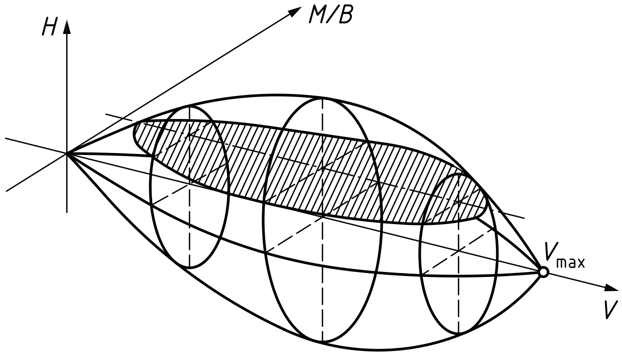


Figure A.6 — Yield surface for drained conditions for a surface foundation[A.7-28]

A.7.3.5.7.2 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.6) by its apex points H_{max}/Q and M_{max}/Q.[A.7-30][A.7-31][A.7-32]

Additional information related to yield surfaces in drained conditions is provided in References [A.7-28] to [A.7-32].

A.7.3.6 Selection of soil parameters for design

A.7.3.6.1 Shear strength used in stability analysis

The following general practices are recommended.

— 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 considered. Specialist geotechnical advice should be sought in these situations.

— In soft and very soft clays, 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.

— Soils display undrained shear strength anisotropy, and thus triaxial compression, triaxial extension and simple shear strengths can be significantly 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.

— 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.

— 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, taking account of action history and average shear stress, is outlined in References [A.7-42].

— 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 References [A.7-43] and [A.7-52]. In undertaking such analyses, it is important to take due account of the effects of drainage with the potential for dissipation of excess pore pressures to occur over the duration of cyclic loading.

— 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.

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

— In soils that display strain softening behaviour, it can be important to consider the effects of strain compatibility when selecting a value of soil strength. This is likely to be of particular importance for skirted foundations, where contributions to stability come from a combination of base shear (at skirt tip level) and passive resistance, which are typically mobilized at very different strain levels.

— Where possible, assessment of appropriate soil parameters should involve statistical treatment of the available data.

A.7.3.6.2 Parameters used in serviceability design

No additional guidance is offered.

A.7.4 Stability of shallow foundations

A.7.4.1 Assessment of bearing capacity

The development of the bearing capacity formulae presented is predicated on the assumption 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 [A.7-33] to [A.7-39]. In general, the formulae and factors outlined in this part of ISO 19901 should be used with care and their applicability should be checked in each case. Where appropriate, alternative methods of analysis and/or design approaches should be considered to verify the results obtained.

Rigorous solutions of bearing capacity factors for perfectly plastic materials (with associated flow) can now be determined by select software programs. Notably, the freeware ABC[A.7-40] 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.

A.7.4.1.1 Undrained conditions (constant shear strength with depth)

A.7.4.1.1.1 Bearing capacity factors

The bearing capacity factor, N_c, for a rigid surface strip foundation with a horizontal base resting on the surface of a horizontal seabed, idealized as a perfectly plastic material of uniform strength under uniaxial vertical action in the absence of other actions is given by Reference [A.7-38].

$$N_c = 2 + \pi = 5,14 \tag{A.8}$$

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.

A.7.4.1.1.2 Bearing capacity correction factors

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

$$K_c = 1 + s_c + d_c - i_c - b_c - g_c \tag{A.9}$$

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:

$$s_c = 0{,}18(1 - 2i_c)\left(\frac{B'}{L'}\right) \tag{A.10}$$

$$d_c = 0{,}3\tan^{-1}\left(\frac{D_b}{B'}\right) \tag{A.11}$$

$$i_c = 0{,}5 - 0{,}5\sqrt{1 - \frac{H_b}{A'\left(\frac{s_u}{\gamma_m}\right)}} \tag{A.12}$$

$$b_c = \frac{2\upsilon}{\pi + 2} \approx 0{,}4\upsilon \tag{A.13}$$

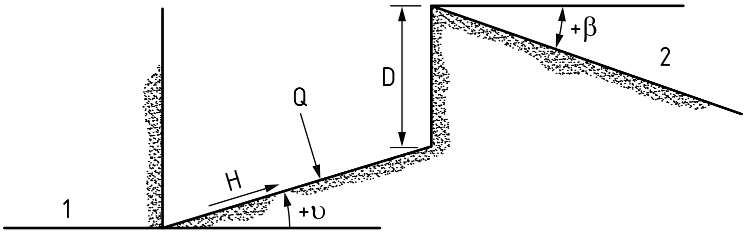
$$g_c = \frac{2\beta}{\pi + 2} \approx 0{,}4\beta \tag{A.14}$$

where

— 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 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.

— υ and β are base and ground inclination angles in radians. Figure A.7 defines these angles for a general foundation problem.



Key

1 horizontal surface

2 ground surface

Figure A.7 — Definitions for inclined base and seafloor surface[A.7-33]

The recommended correction factors s_c, i_c, b_c, and g_c are taken directly from Reference [A.7-33].

The recommended depth factor d_c is taken from Reference [A.7-37] and is slightly more conservative than specified by Reference [A.7-33]. The relevancy of using the above depth factor d_c should be evaluated for individual cases. If the installation procedure and/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 mobilization of significant passive earth pressure between seafloor and foundation base level.

A.7.4.1.2 Undrained conditions (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 [A.7-34].

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

$$F \approx a + bx - \sqrt{\left(c + bx\right)^2 + d^2} \tag{A.15}$$

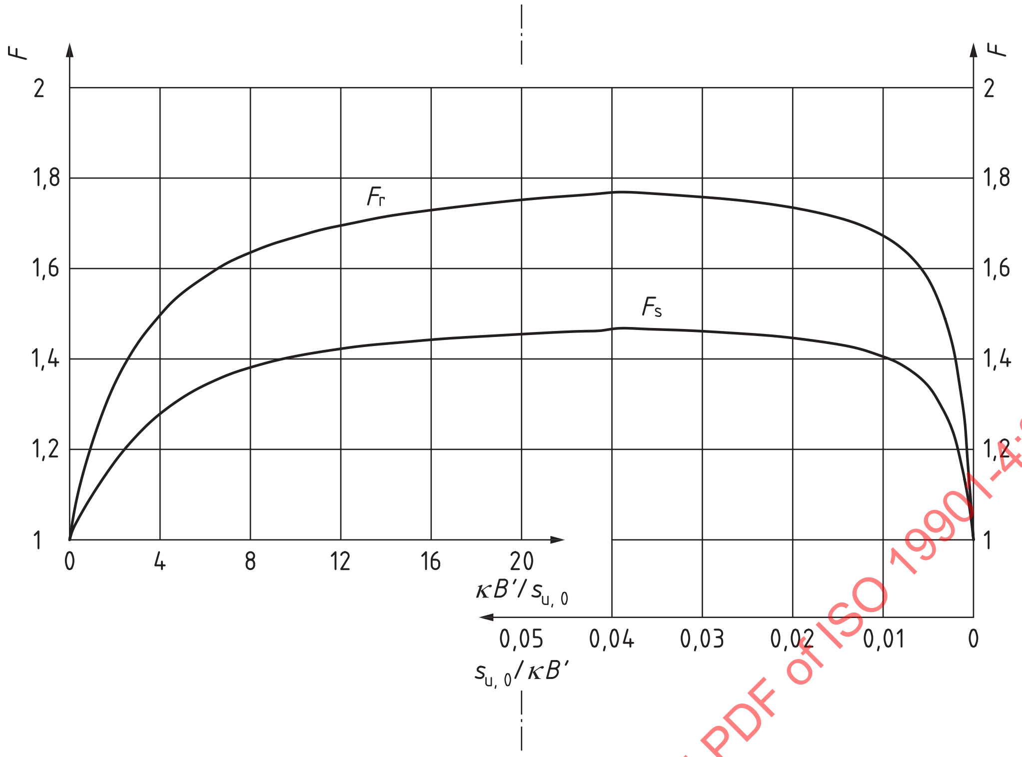
where

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

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.8)

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



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 factor

Figure A.8 — Bearing capacity correction factor *F* for linearly increasing isotropic undrained shear strength with depth[A.7-34]

$$K_c = 1 + s_c + d_c - i_c - b_c - g_c \tag{A.16}$$

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:

$$s_c = s_{cv}(1 - 2i_c)\left(\frac{B'}{L'}\right) \tag{A.17}$$

where s_{cv} is taken as function of $\kappa B'/s_{u0}$ and values of s_{cv} can be approximated using the relationship:

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

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

$$d_c = 0,3\left(\frac{s_{u,1}}{s_{u,2}}\right)\arctan\left(\frac{D_b}{B'}\right) \tag{A.19}$$

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:

$$s_{u,2} = F\frac{\left(N_c s_{u0} + \frac{\kappa B'}{4}\right)}{N_c} \tag{A.20}$$

$$i_c = 0,5 - 0,5\sqrt{1 - \frac{H_b}{A'\left(\frac{s_{u0}}{\gamma_m}\right)}} \tag{A.21}$$

$$b_c = \frac{2\upsilon}{\pi + 2} \approx 0,4\upsilon \tag{A.22}$$

$$g_c = \frac{2\beta}{\pi + 2} \approx 0,4\beta \tag{A.23}$$

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.7.4.1.1.

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 as given in [A.7-37].

The shape factor, s_{cv}, from Reference [A.7-36] 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 above depth factor, d_c, should be evaluated in each case. If the installation procedure and/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 mobilization of significant passive earth pressure between the seafloor and foundation base level.

A.7.4.1.3 Drained conditions

A.7.4.1.3.1 Bearing capacity factors

The following bearing capacity factors given in Formulae (A.24) and (A.25) are recommended for pure vertical action on a strip foundation with no embedment:

$$N_q = \left[\tan\left[\frac{\pi}{4} + 0,5\arctan\left(\frac{\tan\phi'}{\gamma_m}\right)\right]\right]^2 \left[\exp\left(\pi\,\frac{\tan\phi'}{\gamma_m}\right)\right] \qquad \text{(A.24)}$$

$$N_\gamma = 1,5(N_q - 1)\left(\frac{\tan\phi'}{\gamma_m}\right) \qquad \text{(A.25)}$$

Effective friction angles, ϕ', between 30° and 42° are considered reasonable limits for general use with these formulae. Effective friction angles that fall outside of these limits can indicate non-standard soils and should be considered by specialist geotechnical engineers only.

A.7.4.1.3.2 Bearing capacity correction factors

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

$$K_q = s_q d_q i_q b_q g_q \qquad \text{(A.26)}$$

$$K_\gamma = s_\gamma d_\gamma i_\gamma b_\gamma g_\gamma \qquad \text{(A.27)}$$

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.28) to (A.36) for seafloor surface inclination can be unconservative in cases of loose to very loose sand, and specialist advice should be sought in these cases.

Recommended expressions for the correction factors are:

$$s_q = 1 + i_q\left(\frac{B'}{L'}\right)\sin\left[\arctan\left(\frac{\tan\phi'}{\gamma_m}\right)\right] \qquad \text{(A.28)}$$

$$d_q = 1 + 1,2\left(\frac{D_b}{B'}\right)\left(\frac{\tan\phi'}{\gamma_m}\right)\left[1 - \sin\left[\tan^{-1}\left(\frac{\tan\phi'}{\gamma_m}\right)\right]\right]^2 \qquad \text{(A.29)}$$

$$b_q = e^{-2\upsilon\left(\frac{\tan\phi'}{\gamma_m}\right)} \qquad \text{(A.30)}$$

$$g_q = g_\gamma = (1 - 0,5\tan\beta)^5 \qquad \text{(A.31)}$$

$$i_q = 1 - 0,5\left(\frac{H_b}{V_b}\right)^5 \qquad \text{(A.32)}$$

$$s_\gamma = 1 - 0,4 i_\gamma \left(\frac{B'}{L'} \right) \tag{A.33}$$

$$d_\gamma = 1 \tag{A.34}$$

$$i_\gamma = 1 - 0,7 \left(\frac{H_b}{V_b} \right)^5 \tag{A.35}$$

$$b_\gamma = e^{-2,7 \upsilon \left(\frac{\tan \phi'}{\gamma_m} \right)} \tag{A.36}$$

The effective width or effective length is used in the bearing capacity correction factors stated above 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 above depth factor, d_q, should be evaluated in each case. It should be emphasized that 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 and/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, υ and β are as noted in A.7.4.1.1.

A.7.4.1.3.3 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. There has been much debate over whether the deduced c' reflects a true cohesion (or cementation) or is merely an artefact resulting from fitting a tangent to what is, in reality, a curved strength envelope. It has been well established that the friction angle for soils increases as the mean effective stress level decreases, due to increasing dilation. As such, in many cases the effective cementation is 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 [A.7-41].

Examples of where inclusion of an effective cohesion in estimating bearing capacity might be argued as appropriate include:

— Naturally cemented soils (particularly sands). In this case, care is needed because of potentially different mobilization rates for the cemented and frictional components of soil strength, and the possibility that progressive failure might occur, with the cementation reducing to zero before the full frictional strength is mobilized.

— Medium to heavily overconsolidated clays. In this case, ignoring any effective cohesion (or dilation-induced high friction angles at low mean effective stresses) might prove over-conservative. However, inclusion of an effective cohesion in estimating bearing capacity could also prove unduly optimistic, partly because the mean effective stress levels associated with the (drained) bearing capacity might be too high to justify any non-zero effective cohesion, and partly because the level of displacement allowed in design might be too low to fully mobilize the effective cohesion (or dilation-induced high friction angles at low mean effective stresses).

A.7.4.2 Assessment of sliding capacity

No additional guidance is offered.

A.7.4.3 Assessment of torsional capacity

Methods that can be used for assessing torsional stability are provided in Reference [A.7-45] and Reference [A.7-46].

A.7.5 Serviceability (displacements and rotations)

A.7.5.1 General

It should be noted that foundation displacement can be significant at the maximum soil stress levels allowed in this part of ISO 19901, such as where foundation loading reaches the soil yield stress.

A.7.5.2 Displacement under static loading

A.7.5.2.1 General

Static deformations are generally considered to be of two types, (i) immediate deformation, which is the more or less instantaneous response of a foundation to loading and primarily results from shear deformation (shear straining) of the soil; (ii) 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.7.5.2.2 Immediate displacements

Because soils exhibit nonlinear, path dependent behaviour under loading, the short-term deformation problem is quite 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 [A.7-47]. Solutions for rigid, embedded circular foundations are provided in Reference [A.7-48].

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 overconsolidation ratio can lead to satisfactory results. Further discussion of these points is presented in Reference [A.7-49].

Where the foundation base is flexible or the loading is sufficiently severe to create high stresses throughout a significant volume of soil, the expressions provided in this part of ISO 19901 are inappropriate and numerical analyses might be required. Finite element and finite difference techniques have the capability of including complex geometries and loadings and nonlinear, variable soil profiles. Special consideration should be given to the potential effects of softening of the soil (reduction in modulus) under cyclic loading.

A.7.5.2.3 Primary consolidation settlement

Because of the finite extent of the foundation, the vertical stress imposed by the structure should be attenuated with depth. An estimate of such attenuation can be determined from elastic solutions such as those referenced by Reference [A.7-47]. 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 Reference [A.7-50] and

Reference [A.7-51]. If an accurate prediction of rate of settlement is required, 2D or 3D coupled analysis supported by high quality field data are required.

A.7.5.2.4 Secondary compression (creep)

No additional guidance is offered.

A.7.5.2.5 Differential settlements induced by eccentricity

No additional guidance is offered.

A.7.5.3 Displacement under dynamic and cyclic actions

A.7.5.3.1 Foundation response to applied loading

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.

Note that half space solutions can be considerably in error where non-uniform soil profiles exist. In addition, for large amplitude environmental loading nonlinear soil behaviour can be significant. In such cases a numerical analysis might be required or at least a study of a range of soil stiffness properties should be considered.

In some cases it is not appropriate to treat the foundation as an elastic half space, such as where it becomes necessary to model the energy loss in the soil. Specialist advice should be sought in these cases.

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 [A.7-52] and Reference [A.7-53].

A.7.5.3.2 Settlement after the event

No additional guidance is offered.

A.7.5.4 Other contributors to foundation settlement

No additional guidance is offered.

A.7.6 Other design considerations

A.7.6.1 Hydraulic stability

No additional guidance is offered.

A.7.6.2 Installation, retrieval and removal

A.7.6.2.1 General

Shallow foundations are often placed on the seafloor from an installation vessel. The vertical heave motions induced by the vessel's motion characteristics and created by the environmental boundary conditions cause the touchdown to be an impact between the seafloor and the structure. This impact is normally controlled by limiting weather criteria and carefully planned and well controlled installation operations. However, it is still the experience that many small structures suffer a foundation failure during installation, mainly in soft soil conditions. Many such incidents can be avoided by observing the following recommendations.

— If the installation is performed under controlled conditions by use of a heave compensator and low rate of descent towards the seafloor (<0,2 m/s), no extra safety margin should be needed.

— If the installation is performed without any control on the rate of descent (no heave compensation), penetration in excess of the failure displacement can occur. The consequences of such additional penetration should be investigated.

The use of relatively small capacity installation vessels can result in foundation failure; 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 pullout failure in the soil (reverse bearing capacity failure). The soil condition under the structure after such an event is close to a remoulded state and normal partial action and material factors can be too small to prevent foundation failure during final set-down.

If there is a risk of significant heave motions or impact on touchdown, higher material factors than outlined in 7.3 should be applied. Shallow foundations, such as those used for temporary support or for subsea structures, are often designed for limited environmental actions. Since the main actions are permanent and due to gravity, the higher material factor in 7.3 should be applied.

A.7.6.2.2 Skirt penetration resistance

Skirts can provide a significant resistance to penetration. This resistance, Q_r can be estimated as a function of depth by the following:

$$Q_r = Q_f + Q_p = f\,A_s + q\,A_p \tag{A.37}$$

where

Q_f is the skirt friction resistance;

Q_p is the total end bearing resistance from skirt tips;

f is the unit skirt friction;

A_s is the side surface area of skirt embedded at a particular penetration depth (including both internal and external skirt faces);

q is the unit end bearing resistance on the skirt tip;

A_p is the projected area of skirt tip.

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. 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. In most cases, it is highly desirable to achieve full skirt penetration. This should be considered in selecting soil strength properties (or CPT q_c values) for use in analysis as low estimates of strength are non-conservative in this case.

General guidance on assessing skirt penetration based on cone penetrometer resistance can be found in Reference [A.7-37] for North Sea soils and in Reference [A.11-46] for soft deepwater West Africa clays. For non-standard soils, specialist advice can be required.

In general, water will be trapped within the skirt compartments. The penetration rate should be such that removal of the water can be accomplished without forcing it under the skirts and damaging the foundation. In some cases, a pressure drawdown (i.e. negative excess pore pressure or 'suction' relative to ambient pressure) can be used to increase the penetration force, although analysis should be carried out to ensure that it will not result in damage to the foundation soil.

In assessing the penetration of skirts careful attention should be given to site conditions. An uneven seafloor, lateral soil strength variability, existence of boulders, etc. can give rise to uneven penetration or structural damage of skirts. In some cases, site improvements can be required such as levelling the area by dredging or fill emplacement.

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 be substantial but can usually be overcome by sustained uplift forces or by introducing water into the base compartments to relieve the suction.

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

A.8 Pile foundation design

A.8.1 Pile capacity for axial compression

A.8.1.1 General

No 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). This procedure ensures that the pile has an adequate reserve above the design actions in order to accommodate uncertainties in actions and pile resistances.

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 [A.8-11], [A.8-12] and [A.8-13]. For the particular case of a belled pile, this matter is discussed in Reference [A.8-13].

A.8.1.3 Skin friction and end bearing in cohesive soils

A.8.1.3.1 General

Estimating pile capacity in clay soils requires considerable 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

A number of studies, Reference [A.8-3] and References [A.8-14] to [A.8-18], have been carried out and aimed at collecting and comparing axial capacities from relevant pile loading tests to those predicted by traditional offshore pile design procedures. Studies such as these can be very useful in calibrating one's judgment in the design process. It is clear, for example, that there is considerable scatter in the various plots of measured versus predicted capacities. The designer should be aware of the many limitations of such comparisons when making use of these results. Limitations of particular importance include the following.

— There is considerable uncertainty in the determination of both predicted capacities and measured capacities. For example, determination of the predicted capacities is very sensitive to the selection of the undrained shear strength profile, which itself is subject to considerable uncertainty. The measured capacities are also subject to interpretation as well as possible measurement errors.

— The conditions under which the pile loading tests are conducted generally vary significantly from the design actions and field conditions. One clear limitation is the limited number of tests on deeply embedded, large diameter, high capacity piles. Generally, pile loading tests have capacities that are 10 % or less of the prototype capacities. Another factor is that the rate of change and the cyclic history of the actions are usually not well represented in the loading tests (see A.8.3.2). For practical reasons, the pile loading tests are often conducted before full set-up occurs (see A.8.1.3.2.5). Furthermore, the pile tip conditions (closed versus open-ended) can differ from offshore piles.

— In most of the studies, an attempt has been made to eliminate those tests that are thought to be significantly affected by extraneous conditions in the loading test, such as protrusions on the exterior of the pile shaft (weld beads, cover plates, etc.), installation effects (jetting, drilled out plugs, etc.), and artesian conditions. Nevertheless, it is not possible to eliminate all extraneous factors in all cases.

The database includes a number of tests that were specially designed for offshore applications as well as a number of published tests that are fortuitously relevant to offshore conditions (appropriate pile type, installation method, soil conditions, etc.). The former are generally higher quality and larger scale, and hence are particularly important in calibrating the design method. The tests most relevant to offshore applications have all been conducted in the United States or in Europe. As regional geology and particularly operating experience are considered very important in foundation design, care should be exercised in applying these results to other regions of the world. In addition, the designer should note that certain important tests in silty clays of low plasticity, such as at the Pentre site,[A.8-19] indicate overprediction of frictional resistance by the Formulae (21) to (23). The reason for this overprediction is not well understood and has been an area of active research. The designer is thus cautioned that pile design for soils of this type should be given special consideration.

Additional considerations that apply to drilled and grouted piles are discussed in References [A.8-20] to [A.8-21].

A.8.1.3.2.2 Alternative methods of determining pile capacity

Alternative methods of determining pile capacity in clays exist, based on sound engineering principles and consistent with industry experience, and can be used in practice. One such method is described below.

For piles driven through clay, $f(z)$ can be less than or equal to, but should not exceed, the undrained shear strength of the clay, s_u, as determined by unconsolidated-undrained (UU) triaxial tests and miniature vane shear tests or the following limits:

a) for highly plastic clays, $f(z)$ can be equal to s_u for underconsolidated and normally consolidated clays. For overconsolidated clays, $f(z)$ should not exceed 48 kPa (1 kips/ft^2) for shallow penetrations or the equivalent value of s_u for a normally consolidated clay for deeper penetrations, whichever is greater;

b) for other types of clay,

$$f(z) = s_u \text{ for } s_u < 24 \text{ kPa } (0{,}5 \text{ kips/ft}^2) \tag{A.38}$$

$$f(z) = s_u / 2 \text{ for } s_u > 72 \text{ kPa } (1{,}5 \text{ kips/ft}^2) \tag{A.39}$$

$f(z)$ varies linearly for values of s_u between the above limits.

For other methods, see References [A.8-4], [A.8-14], [A.8-15] and [A.8-17].

It has been shown in Reference [A.8-18] that, on average, the above cited methods predict the available but limited pile loading test database results with comparable accuracy. However, capacities for specific situations computed by different methods can differ by a significant amount. In such cases, pile capacity determination should be based on engineering judgment, which takes into account site-specific soils information, available pile loading test data, and industry experience in similar soils.

A.8.1.3.2.3 Establishing design strength and effective overburden stress profiles

The axial pile capacity in clay determined by these procedures 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 the potentially large scatter in the strength data from the various types of laboratory tests complicate appropriate selection. ISO 19901-8 provides additional information on the sampling and laboratory testing techniques and on the quality of marine soil investigations.

UU triaxial compression tests on high quality samples, preferably taken by pushing a thin-walled sampler with a diameter of 75 mm (3 in) or more into the soil, are recommended for establishing strength profile variations because of their consistency and repeatability. In selecting the specific shear strength values for design, however, consideration should be given to the sampling and testing techniques used to correlate the shear strength to any available relevant pile loading test data. The experience with pile performance is another consideration that can play an important role in assessing the appropriate shear strength interpretation.

Miniature vane tests on the pushed samples should correlate well with the UU test results and will be particularly beneficial in weak clays. *In situ* testing with a vane or penetrometers (such as the cone, ball and T-bar) will help in assessing sampling disturbance effects in gassy or highly structured soils. Penetrometer data will furthermore help in determining a continuous design strength profile. Approaches such as the SHANSEP technique (Stress History and Normalized Soil Engineering Properties, see References [A.8-22] and [A.8-23]) can help provide a more consistent interpretation of standard laboratory tests and can provide history information used to determine the effective overburden stress in normally or underconsolidated clays.

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;

— lateral movement of soil away from the pile due to pile ‘whip’ during driving.

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 References [A.8-11] to [A.8-15] and Reference [A.8-17].

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 the guidelines given in 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 cohesive 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 cohesive deposit can be relatively long, as much as two to three years.[A.8-19][A.8-24][A.8-25] However, it should be noted that 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 overconsolidated 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 overconsolidated clays.[A.8-26][A.8-27] 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 overconsolidated 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.

In Reference [A.8-24], the behaviour of piles subjected to significant axial actions in highly plastic, normally consolidated clays was studied using a large number of model pile tests and some full-scale pile loading tests. From the study of pore pressure dissipation and loading test data at different times after pile driving, empirical correlations were obtained between the degree of consolidation, degree of plugging, and pile shaft shear transfer capacity. The analysis is dependent on the shear strength of the surrounding soil mass. The method is presently limited to use in highly plastic, normally consolidated clays of the type encountered in the Gulf of Mexico, since validation data have been published only for those soils.

In Reference [A.8-25], in highly overconsolidated glacial till, capacity was shown to undergo significant short-term reduction associated with pore pressure redistribution and reduction in radial effective stresses during the early stages of the equalization process. The capacity at the end of installation was never fully recovered. Test results for closed-ended steel piles in heavily overconsolidated London clay indicate that there is no significant change in capacity with time.[A.8-28] This is contrary to tests on 0,273 m (10,75 in) diameter closed-ended steel piles in overconsolidated Beaumont clay, where considerable and rapid set-up (in four days) was found.[A.8-29]

Caution should be exercised in using this subclause to evaluate set-up, particularly for soils with different plasticity characteristics and under different states of consolidation (especially overconsolidated clays) and piles with D/WT (pile outer diameter/pile wall thickness) ratios greater than 40.

A.8.1.4 Skin friction and end bearing in cohesionless soils

A.8.1.4.1 General

Estimating axial pile capacity in cohesionless soils requires considerable engineering judgment in selecting an appropriate method and associated parameter values. Some of the items that should be considered by geotechnical engineers are detailed in A.8.1.4.2.

A.8.1.4.2 discusses four CPT-based methods for axial pile capacity that incorporate length effects and friction fatigue. Some of these methods, with some offshore experience given in References [A.8-46] to [A.8-49], have not yet been frequently compared for routine offshore pile projects. Hence, geotechnical engineering judgment is needed to select the most appropriate method for the design case under consideration. Additional care is required in cases of clay layers at or near pile tip level.

The piles are assumed to be open-ended steel piles of uniform outer diameter. Installation is by impact driving into significant depths of clean siliceous sand. In general, such piles drive unplugged (i.e. they core). However, when they are statically loaded in compression, sufficient inner friction is generally mobilized to cause the pile to act as fully plugged (i.e. the soil plug does not undergo overall slip relative to the pile wall during compression pile loading).

The term *sand* is used hereafter for all cohesionless siliceous soils. Exceptions are addressed in A.6.4 (carbonate sands) and in A.8.1.4.2.7 (gravels).

The appropriate resistance factors to be used with the methods discussed in A.8.1.4.2 are not provided in A.8.1.4. The designer should carefully evaluate, for each design case, whether the resistance factors provided in ISO 19902 are appropriate or not.

Following installation, pile driving (instrumentation) data can be used to give more confidence in predicted capacities. Additional guidance is given in Clause 9 and A.9.

A.8.1.4.2 CPT-based methods for pile capacity

A.8.1.4.2.1 General

In 8.1.4 a simple method for assessing pile capacity in cohesionless soils is presented, which is a modification of methods recommended in the past. Loading test data for piles in cohesionless soils indicate that variability in capacity predictions using the simple method described in 8.1.4 can exceed those for piles in clay.[A.8-1][A.8-35] These data also indicate that the method described in 8.1.4 is conservative for short offshore piles (<45 m; 150 ft) in dense to very dense sands loaded in compression and can be unconservative in all other conditions. Therefore, in situations where unconservative results are expected or in unfamiliar situations, the designer should account for this uncertainty through the selection of conservative design parameters and/or higher material or resistance factors. This is especially important where force redistributes after the development of maximum resistance occurs, which can lead to an abrupt (brittle) failure — such as is the case for short piles in tension. Changes were made to remove potential unconservatism.

A.8.1.4.2.1 to A.8.1.4.2.5 present recent and more reliable CPT-based methods for predicting pile capacity. These methods are all based on direct correlations of pile unit friction and end bearing data with cone tip resistance values from cone penetration tests (CPT).

Friction and end bearing contributions to pile capacity are assumed to be uncoupled. Hence, for all methods, the representative value of the axial pile capacity in compression ($Q_{r,c}$) and in tension ($Q_{r,t}$) of plugged open-ended piles is determined by:

$$Q_{r,c} = Q_{f,c} + Q_p = \pi\ D \int f_c(z)\,dz + q \cdot A_{pile} \tag{A.40}$$

$$Q_{r,t} = Q_{f,t} = \pi\ D \int f_t(z)\,dz \tag{A.41}$$

where

- $Q_{r,c}$ is the representative value of the axial pile capacity in compression (in force units);
- $Q_{r,t}$ is the representative value of the axial pile capacity in tension (in force units);
- $Q_{f,c}$ is the representative value of the total skin friction resistance in compression (in force units);
- $Q_{f,t}$ is the representative value of the total skin friction resistance in tension (in force units);
- Q_p is the representative value of the end bearing capacity (in force units);
- $f_c(z)$ is the unit skin friction in compression, which is a function of depth, geometry and soil conditions (in stress units);
- $f_t(z)$ is the unit skin friction in tension, which is a function of depth, geometry and soil conditions (in stress units);
- z is the depth below the original seafloor (m);

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

D is the pile outside diameter (m);

A_{pile} is the gross end area of the pile, $A_{pile} = \pi D^2/4$ (m^2).

Since the friction component, Q_f, involves numerical integration, results are sensitive to the depth increment used, particularly for CPT-based methods. As guidance, depth increments for CPT-based methods should be in the order of 1/100 of the pile length (or smaller). In any case, the depth increment should not exceed 0,2 m (0,5 ft).

The four recommended CPT-based methods discussed herein are:

— method 1 Simplified ICP-05 (as described in this part of ISO 19901);

— method 2 Offshore UWA-05;[A.8-30][A.8-31]

— method 3 Fugro-05;[A.8-30][A.8-32] and

— method 4 NGI-05.[A.8-30][A.8-33]

Method 1 is a simplified version of the design method recommended in Reference [A.8-34], whereas method 2 is a simplified version of the UWA-05 method applicable to offshore pipe piles. Methods 2, 3 and 4 are summarized in Reference [A.8-30]. Friction and end-bearing contributions should not be taken from different methods. A general description of methods 1, 2 and 3 is given hereafter, after which details of the various methods are presented separately.

The unit skin friction formulae for open-ended steel pipe piles for CPT-based methods 1, 2 and 3 can all be considered as being special cases of the general formula:

$$f(z) = u\, q_c(z) \left[\frac{\sigma'_{v0}(z)}{p_a}\right]^a A_r^b \left[\max\left(\frac{L-z}{D}, v\right)\right]^{-c} \left(\tan\delta_{cv}\right)^d \left[\min\left(\frac{L-z}{D}\cdot\frac{1}{v}, 1\right)\right]^e \quad \text{(A.42)}$$

where, in addition to prior definitions

$f(z)$ is the unit skin friction, which is a function of depth, geometry and soil conditions (in stress units);

$q_c(z)$ is the CPT cone-tip resistance at depth, z (in stress units);

$\sigma'_{v0}(z)$ is the effective vertical stress of the soil at depth, z (in stress units);

p_a is the atmospheric pressure, e.g. $p_a = 100$ kPa;

A_r is the pile displacement ratio, $A_r = A_w/A_{pile} = 1 - (D_i/D)^2$ (-), where A_w is the area of the rim of the steel pile, $A_w = (\pi/4)\cdot(D^2 - D_i^2)$ (m^2), D_i is the pile inside diameter, $D_i = D - 2\,WT$(m), and WT is the pile wall thickness (m);

L is the embedded length of pile below original seafloor (m);

δ_{cv} is the constant volume friction angle at the interface between the soil and the pile wall.

Recommended values for the parameters, a, b, c, d, e, u and v, for compression and tension are given in Table A.3.

Additional recommendations for computing unit friction and end bearing of all four CPT-based methods are presented in A.8.1.4.2.7.

Table A.3 — Unit skin friction parameter values for driven open-ended steel piles for methods 1, 2 and 3

Method	Parameter a	b	c	d	e	u	v
Method 1:							
Compression	0,1	0,2	0,4	1	0	0,023	$4\sqrt{A_r}$
Tension	0,1	0,2	0,4	1	0	0,016	$4\sqrt{A_r}$
Method 2:							
Compression	0	0,3	0,5	1	0	0,030	2
Tension	0	0,3	0,5	1	0	0,022	2
Method 3:							
Compression	0,05	0,45	0,90	0	1	0,043	$2\sqrt{A_r}$
Tension	0,15	0,42	0,85	0	0	0,025	$2\sqrt{A_r}$

A.8.1.4.2.2 Method 1

a) Friction

Reference [A.8-34] presents a comprehensive overview of research work performed at Imperial College on axial pile design criteria of open- and closed-ended piles in clay and sand. The design formulae for unit friction in sand in this reference include a soil dilatancy term, implying that unit friction is favourably influenced by soil dilatancy. This influence diminishes with increasing pile diameter. The unit skin friction $f(z)$ of open-ended pipe piles in method 1, given by Formula (A.42) and the parameter values in Table A.3, are a conservative approximation of the full calculation in method 1, since dilatancy is ignored and some parameter values were conservatively rounded up/down.

The original 'full' design formulae in Reference [A.8-34] can be used, particularly for small diameter piles [$D < 0{,}76$ m (30 in)], provided that larger resistance factors are considered. See Reference [A.8-34] and Reference [A.8-47] for a discussion on reliability-based design using the 'full' method 1.

b) End bearing

The unit end bearing, q, for open-ended pipe piles follows the recommendations of Reference [A.8-34]. These specify a unit end bearing for plugged piles given by:

$$q = q_{\mathrm{c,av,1,5D}} \left[0{,}5 - 0{,}25 \log_{10}\left(\frac{D}{D_{\mathrm{CPT}}} \right) \right] \geq 0{,}15\, q_{\mathrm{c,av,1,5D}} \tag{A.43}$$

where, in addition to the general definitions given in A.8.1.4.2.1,

$q_{\mathrm{c,av,1,5D}}$ is the average value of $q_c(z)$ between 1,5 D above the pile tip and 1,5 D below the pile tip.

$$q_{\mathrm{c,av,1,5D}} = \left[\int_{L-1,5D}^{L+1,5D} q_{\mathrm{c}}(z)\,\mathrm{d}z \right] \Big/ (3\,D) \tag{A.44}$$

D_{CPT} is the diameter of the CPT tool, D_{CPT} = 36 mm for a standard cone with a base area of 1 000 mm^2 (see ISO 19901-8).

Reference [A.8-34] specifies that plugged pile end bearing capacity applies, which means that the unit end bearing, q, acts across the entire pile tip cross-section, provided both of the following conditions are met:

$$D_i < 2\ (D_r - 0{,}3) \tag{A.45}$$

$$D_i \;/\; D_{CPT} < 0{,}083\ \frac{q_c(z)}{p_a} \tag{A.46}$$

where D_r is the relative density of the sand ($0 \le D_r \le 1{,}0$).

NOTE Formula (A.45) is the formula given in Reference [A.8-34]. It implies that the maximum pile inside diameter for plugged behaviour in method 1 is 1,4 m. However, experience indicates that offshore piles with appreciably larger inside diameters can still behave as plugged.

If either of the above conditions is not met, then the pile will behave unplugged and Formula (A.47) should be used for computing the end bearing capacity:

$$Q_p = \pi\ (D - WT)\ WT\, .\, q_{c,tip} \tag{A.47}$$

where $q_{c,tip}$ is the cone resistance at the pile tip.

The full pile end bearing computed using Formula (A.43) for a plugged pile should not be less than the end bearing capacity of an unplugged pile computed in accordance with Formula (A.47).

A.8.1.4.2.3 Method 2

a) Friction

Reference [A.8-30] summarizes the results of research work at the University of Western Australia on development of axial pile design criteria for open- and closed-ended piles driven into silica sands. The full design method (described in References [A.8-30] and [A.8-31]) for unit friction on pipe piles includes a term allowing for favourable effects of soil dilatancy (similar to method 1) and an empirical term allowing for partial soil plugging during pile driving. The authors of Reference [A.8-31] recommend for offshore pile design to ignore these two favourable effects (dilatancy and partial plugging), resulting in the recommended Formula (A.42) and associated Table A.3 parameter values.

The original 'full' design formulae in Reference [A.8-30] can be used, particularly for small diameter piles [D < 0,76 m (30 in)], provided that larger resistance factors are considered. See Reference [A.8-30] for a discussion on reliability-based design using method 2.

b) End bearing

References [A.8-30] and [A.8-31] present design criteria for representative unit end bearing of plugged open-ended pipe piles. Their 'full' design method for pipe piles includes an empirical term allowing for the favourable effect of partial plugging during pile driving. For offshore pile design, References [A.8-30] and [A.8-31] recommend to ignore this effect, resulting in the recommended design formulae for plugged piles in method 2:

$$q = q_{c,av,1,5D}\ (0{,}15 + 0{,}45\ A_r) \tag{A.48}$$

where, again in addition to the general definitions given in A.8.1.4.2.1

$q_{c,av,1,5D}$ is the average value of $q_c(z)$ between 1,5 D above the pile tip and 1,5 D below the pile tip.

$$q_{c,av,1,5D} = \left[\int_{L-1,5D}^{L+1,5D} q_z(z)\,\mathrm{d}z\right] \Big/ (3\ D)$$

Since method 2 considers non-plugging under static loading to be exceptional for typical offshore piles, the method does not provide criteria for unplugged piles. The unit end bearing q calculated in Formula (A.40) is therefore acting across the entire tip cross-section. The use of $q_{\mathrm{c,av,1,5D}}$ in Formula (A.48) is not recommended in sand profiles where the CPT qc values show significant variations in the vicinity of the pile tip or when penetration into a founding stratum is less than five pile diameters. For these situations, Reference [A.8-31] provides guidance on the selection of an appropriate average q_{c} value for use in place of $q_{\mathrm{c,av,1,5D}}$.

A.8.1.4.2.4 Method 3

a) Friction

Method 3 is a modification of method 1.[A.8-35] The unit friction formulae were unfortunately misprinted in References [A.8-32] and [A.8-35] and these references are not to be used in design. However, the correct formulae are presented both by Reference [A.8-30] and by Formula (A.42) and the parameter values in Table A.3. Like the ‘full’ method 1 and the ‘full’ method 2, it is recommended that larger resistance factors are considered when using method 3. See Reference [A.8-36] for a discussion on reliability-based design using method 3.

b) End bearing

The basis for the representative unit end bearing on pipe piles in accordance with method 3 is presented in Reference [A.8-35] and summarized in Reference [A.8-32]. The recommended value of the unit end bearing for plugged piles is given by:

$$q = 8{,}5\, p_{\mathrm{a}} \left(\frac{q_{\mathrm{c,av,1,5D}}}{p_{\mathrm{a}}} \right)^{0{,}5} A_{\mathrm{r}}^{\;0{,}25} \qquad \text{(A.49)}$$

where, again in addition to the general definitions given in A.8.1.4.2.1,

$q_{\mathrm{c,av,1,5D}}$ is the average value of $q_{\mathrm{c}}(z)$ between 1,5 D above the pile tip and 1,5 D below the pile tip.

Both method 2 and method 3 do not specify unplugged end bearing capacity because typical offshore piles behave in a plugged mode during static loading.[A.8-36] It can be shown that plugged behaviour applies if either

— the cumulative thickness of sand layers within a soil plug is in excess of 8 , or

— the total end bearing, Q_{p}, is limited as follows:

$$Q_{\mathrm{p}} \leq Q_{\mathrm{f,i,clay}}\, e^{L_{\mathrm{s}}/D} \qquad \text{(A.50)}$$

where

$Q_{f,i,clay}$ is the representative value of the cumulative skin friction resistance of the clay layers within the soil plug (in force units);

L_s is the length of the plug in the sand layers (m).

The cumulative frictional capacity of the clay layers within the soil plug, $Q_{f,i,clay}$, can be estimated using similar procedures as for computing estimated pile friction in clay (see 8.1.3).

Formula (A.50) applies for fully drained behaviour of sand within the pile plug. Criteria for undrained/partially drained sand plug behaviour are presented in Reference [A.8-37].

For the exceptional case of unplugged end bearing behaviour in fully drained conditions, reference is made to References [A.8-36] and [A.8-38] for estimating end bearing capacity.

A.8.1.4.2.5 Method 4

a) Friction

Representative unit skin friction values for tension $f_t(z)$ and compression $f_c(z)$ for driven open-ended steel pipe piles in method 4 are given in Reference [A.8-33].

$$f_t(z) = (z/L)\, p_a\, F_{sig}\, F_{Dr} > 0{,}1\sigma'_{v0}\,(z) \tag{A.51}$$

$$f_c(z) = 1{,}3\,(z/L)\, p_a\, F_{sig}\, F_{Dr} > 0{,}1\sigma'_{v0}\,(z) \tag{A.52}$$

where, again in addition to the general definitions given in A.8.1.4.2.1

$$F_{sig} = (\sigma'_{v0}\,(z)/p_a)^{0{,}25} \tag{A.53}$$

$$F_{Dr} = 2{,}1\,(D_r - 0{,}1)^{1{,}7} \tag{A.54}$$

$$D_r = 0{,}4 \times \ln\left(\frac{q_c(z)}{22\cdot\left(\sigma'_{v0}(z)\,p_a\right)^{0{,}5}}\right) \geq 0{,}1 \tag{A.55}$$

Values of $D_r > 1$ should be accepted and used.

Like for the ‘full’ methods 1, 2 and 3, higher resistance factors should be considered when using method 4.

b) End bearing

The recommended formula for the representative unit end bearing of plugged open-ended steel pipe piles in method 4[A.8-33] is:

$$q = \frac{0{,}7\, q_{c,av,1,5D}}{1 + 3\, D_r^2} \tag{A.56}$$

where

$q_{c,av,1,5D}$ is the average value of $q_c(z)$ between 1,5 D above the pile tip and 1,5 D below the pile tip;

D_r is the relative density of the sand and is derived as described in Formula (A.55).

Values of $D_r > 1$ should be accepted and used.

The resistance of non-plugging piles should be computed using a unit end bearing value for the steel pile rim, $q_w(z)$, given by:

$$q_w(z) = q_c(z) \tag{A.57}$$

and unit friction, $f_p(z)$, between the soil plug and the inner pile wall given by:

$$f_p(z) = 3\, f_c(z) \tag{A.58}$$

The lower of the plugged resistance, q, of Formula (A.56) and the unplugged resistance determined by Formulae (A.57) and (A.58) should be used in design.

A.8.1.4.2.6 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 d).

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.

It should be noted that the CPT-based design methods were established for cone resistance values, q_c, up to 100 MPa. Caution should be exercised when applying the enclosed methods to sands 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, one can consider overlapping CPT push strokes.

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 carefully 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) need to be modelled conservatively.

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.

b) Parameter, $\sigma'_{v0}(z)$

Usually, pore water pressures in sands are hydrostatic and in this case σ'_{v0} 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) Parameter, D_r

Common practice is to use the relationship for Ticino-Toyura-Hokksund sand between q_c and D_r as proposed by Reference [A.8-39].

$$D_r = \frac{1}{2,96} \ln \left(\frac{q_c(z) / p_a}{24,94 \left(\dfrac{p'_m(z)}{p_a} \right)^{0,46}} \right) \tag{A.59}$$

where

$p'_m(z)$ is the effective mean *in situ* soil stress at depth z, $p'_m(z) = (1/3)\ [\sigma'_{v0}(z) + 2\ \sigma'_{h0}(z)]$, where $\sigma'_{h0}(z)$ is the effective horizontal *in situ* stress at depth z;

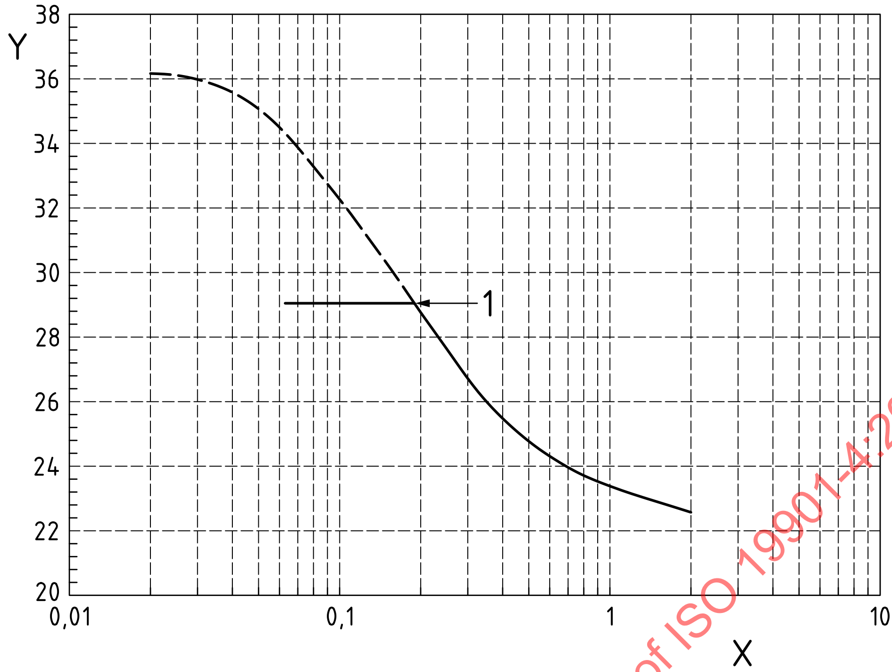
p_a is the atmospheric pressure, in same units as $p'_m(z)$ and $q_c(z)$, e.g. p_a = 100 kPa.

Ticino sand is a medium-grained silica sand with no fines. A reasonably comprehensive database is available for this sand.[A.8-40] However, D_r assessment for method 4 should be in accordance with Formula (A.55). Most $q_c(z)$–D_r relationships are not valid for silty sands. However, $q_c(z)$ can be adjusted for such materials to derive a 'clean sand equivalent normalized cone resistance' (see, for example, References [A.8-41] and [A.8-42]).

d) Parameter, $\tan\delta_{cv}$

The constant volume interface friction angle, δ_{cv}, should be measured directly in laboratory interface shear tests. The recommended test method is by ring shear apparatus, but the direct shear box can also be used. Guidance on test procedures is provided in Reference [A.8-34] and in ISO 19901-8.

If site-specific tests cannot be performed, the constant volume interface friction angle can be estimated as a function of mean effective particle diameter, D_{50}, using Reference [A.8-34]. An upper limit of $\tan\delta_{cv}$ = 0,55 (δ_{cv} = 28,8°) applies to all methods as shown in Figure A.9. However, for materials with unusually weak grains or compressible structures this method is not always appropriate. Of particular importance are sands containing calcium carbonate, for which specific advice is given in A.6.4.



Key

1 recommended upper limit tan $\delta_{cv} = 0{,}55$

X mean particle diameter D_{50} (mm)

Y δ_{cv} (°)

δ_{cv} is the sand constant volume friction angle at the interface between the sand and the pile wall

Figure A.9 — Interface friction angle in sand, δcv, from direct shear interface tests[A.8-34]

A.8.1.4.2.7 Application of CPT-based methods

A.8.1.4.2.2 to A.8.1.4.2.5 provide four methods for computing pile capacity in silica sands using CPT data. Items a) to g) give guidance on the following aspects of pile design using CPT-based methods:

— axial resistance–displacement behaviour;

— application to soils other than silica sands;

— application to piles with different geometries than typical offshore piles;

— effects of scour on pile capacity.

a) *t–z* data for axial shear transfer–displacement response

No strain softening is applicable. However, unlike for the simplified method given in Formula (A.42), the peak unit skin frictions in compression and tension at given depths, $f_c(z)$ and $f_t(z)$, are not unique and both depend on pile geometry. They depend not only on the pile diameter and wall thickness, but also on the total pile penetration. An increased pile penetration will decrease these values at a given depth.

b) *q–z* data for end bearing–displacement response

Unit end bearing, q, is assumed to be fully mobilized at a pile tip displacement value of 0,1D. This displacement is consistent with the manner in which pile loading test data are interpreted.

c) Application to sands other than siliceous sands

Sands other than siliceous sands include carbonate sands, micaceous sands, glauconitic sands, volcanic sands, silts and clayey sands. Some cohesionless soils have unusually weak structures/compressible grains, such as carbonate sands and silts. These generally require special *in situ* and/or laboratory tests for selection of an appropriate design method with associated design parameters. Consideration should be given to using a design method for clays for cases of low permeability sands and silts. All former methods should be applied cautiously since limited data are available to support their reliability in these sediments.[A.8-43][A.8-44][A.8-45]

d) Cone resistance $q_c(z)$ in gravel

The measured $q_c(z)$ data should not be taken at face value in gravel and appropriate adjustments should be made. For example, cone penetration tests made in (coarse) gravels, especially when particle sizes are in excess of 10 % of the CPT cone diameter, are misleading and one possible approach could be to use the lower bound $q_c(z)$ profile. Alternatively, one can estimate an appropriate design $q_c(z)$ profile from adjacent sand layers.

e) End bearing Q_p in presence of weaker clay layers near pile tip

The $q_c(z)$ data used can have a substantial impact on the fully plugged unit end bearing q. The use of $q_c(z)$ data averaged between 1,5 D above the pile tip to 1,5 D below the pile tip level should generally be satisfactory, providing $q_c(z)$ does not vary significantly. This is not necessarily the case when clay layers occur. If significant $q_c(z)$ variations occur, then Figure 2.2 of Reference [A.8-30] should be used to compute a suitable average $q_{c,av}$ value. Alternatively, end bearing assessment in layered soils with adjacent weaker layers can be based on lower bound CPT traces as the pile base can ‘sense’ the presence of weaker layers for a considerable distance below the tip.

Thin clay layers (less than around 0,1 D thick) are problematic, particularly when CPT data are discontinuous vertically and/or not all pile locations have been investigated. Factors to be considered include the variance of layer thickness and of strength and compression parameters. If no direct data are available, a cautious interpretation should be made based on the engineering geology of the surrounding sand soil unit. Offshore piles usually develop only a small percentage of end bearing under extreme conditions. Hence, capacity and settlement calculations, using a finite element model of a pile tip on sand containing weaker layers, should be considered to adequately assess axial pile response under such conditions.

For thick clay layers, shallow geophysical data can be useful to assess layer thickness and elevation. The recommendation in 8.1.4 is to reduce the end bearing component if the pile tip is within a zone up to ±3 D from such layers. If averaging of $q_c(z)$ data are applied to this ±3 D zone, the combined effects can be unduly cautious and such results should be critically reviewed. Similarly, for large diameter piles (say $D > 2$ m), the reduction method in 8.1.4 should be carefully reviewed.

f) Near-shore and onshore piles

In general, for near-shore and onshore piles, the assumptions in A.8.1.4.1 and A.8.1.4.2 are not necessarily valid and should be checked.

Near-shore and onshore pipe piles can respond unplugged when loaded due to insufficient mobilization of inner friction. Similarly, dilatancy effects (which are neglected for offshore piles) can be considered for smaller diameter piles. Scour (especially general scour) can be significant for near-shore pile foundations. In addition, driven closed-ended instead of open-ended steel piles are sometimes used.

The original publications of References [A.8-30], [A.8-33], [A.8-34] and [A.8-36] should be consulted for assumptions made and for further guidance; most of these references include methods to provide the capacity of unplugged pipe piles and of closed-ended piles.

g) 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. Publications such as Reference [A.8-50] give techniques for scour depth assessment. In addition, general scour data can be obtained from national authorities.

In lieu of project specific data, A.8.5.6 gives advice on local scour depth.

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 [A.8-51] recommends that $q_c(z)$ is simply proportional to $\sigma'_{v0}(z)$, i.e.:

$$q_{c,f}(z) = \chi\, q_{c,0}(z) \tag{A.60}$$

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 (Reference [A.8-52]) can be used to determine χ from:

$$\chi = \frac{1}{1+2K_0}\sqrt{\frac{z'+2K_0\sqrt{\Delta z_{GS}\times z'+z'^2}}{\Delta z_{GS}+z'}} \tag{A.61}$$

where

Δz_{GS} is the general scour depth (m);

z' is the final depth below seafloor, after general scour, ($z' = z - \Delta z_{GS}$)(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.8.5 gives a method to reduce the effective stress, σ'_v, for both general and local scour.

A.8.1.5 Skin friction and end bearing of grouted piles in rock

No additional guidance is offered.

A.8.2 Pile capacity for axial tension

No additional guidance is offered.

A.8.3 Axial pile performance

A.8.3.1 Static axial behaviour of piles

An analytical method for determining axial pile performance is provided in Reference [A.8-52]. This method makes use of *t–z* curves of local transfer of axial pile shear, *t*, against local pile displacement,

z, to model the axial support provided by the soil along the side of the pile. An additional Q–z curve is used to model the tip end bearing, Q, against tip displacement, z. Methods for constructing t–z and Q–z curves are given in 8.4.

In some circumstances, i.e. for soils that exhibit strain-softening behaviour and/or where the piles are excessively axially flexible, the actual capacity of the pile can be less than the ultimate capacity given by Formula (20). In these cases, an explicit consideration of these effects on axial capacity is warranted.

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. On the other hand, extreme care should be exercised in applying this approach. 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.8-53]

A.8.3.2.3 Cyclic effects

Detailed consideration of cyclic effects can be warranted 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:

— decrease capacity and stiffness due to repeated actions,[A.8-54] or

— increase capacity and stiffness due to high rates of change of actions.[A.8-55]

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 References [A.8-53] to [A.8-58].

A.8.3.2.4 Analytical models

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.8.3.2.4.1 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 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 Reference [A.8-59] to [A.8-61]. 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.[A.8-62] 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.8.3.2.4.2 Continuum models

The soil around the pile is idealized as a continuum attached continuously to the pile. The material behaviour can incorporate virtually any reasonable stress-strain rules the analyst can devise. Depending on the degree of nonlinearity and heterogeneity, this model can be quite complicated. Again, the pile is typically modelled as a continuous rod, either linear or nonlinear. In these models material properties can vary in any direction.[A.8-63][A.8-64]

There is a wide range of assumptions that can be used regarding boundary conditions, solution characteristics, etc., which lead to an unlimited number of variations for either of the two approaches.

Once the idealized model is established and the relevant formulae are developed, then 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.[A.8-53][A.8-60] Classical finite element models have been used for specialized analyses of piles subjected to monotonic axial actions.[A.8-63]

For practical reasons, discrete element models, solved numerically, have seen the most use in evaluation of piles subjected to high intensity cyclic action. Results from these models are used to develop information on pile accumulated displacements and on pile capacity following high intensity cyclic actions.[A.8-60][A.8-61]

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.[A.8-63][A.8-64] 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.[A.8-65]

A.8.3.2.5 Soil characterization

A key part of developing realistic analytical models to evaluate cyclic effects on piles is the characterization of soil-pile interaction behaviour. High quality *in situ*, laboratory and model-prototype pile loading tests are essential in such characterizations. In developing (soil) characterizations relevant for soil-pile interaction, it is important that pile installation and relevant conditions of the actions on a pile be integrated into the testing programmes.[A.8-53][A.8-61]

In situ tests (e.g. vane shear, cone penetrometer, Ball or T-bar penetrometer, pressuremeter) can provide important insights into in-place soil behaviour and stress-strain properties.[A.8-42][A.8-66] Both low and high amplitude stress-strain properties can be developed. Long-term (static, creep), short-term (dynamic, impulsive) and cyclic (repeated) actions sometimes can be simulated with *in situ* testing equipment.

Laboratory tests on representative soil samples permit a wide variety of stress-strain conditions to be simulated and evaluated.[A.8-67] 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 important source of data for developing soil characterizations for cyclic analyses. Model piles can be highly instrumented and repeated tests can be performed in soils and for a variety of actions.[A.8-61][A.8-68] Geometrical scale, time scale and other modelling effects should be carefully considered in applying results from model tests to analyses of prototype behaviour.

Data from loading tests on prototype piles are useful for calibrating analytical models.[A.8-69][A.8-70][A.8-71][A.8-72] Such tests, even if not highly 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 References [A.8-53], [A.8-61], [A.8-62], [A.8-73] and [A.8-74]. Prototype pile loading tests coupled with *in situ* and laboratory soil testing and realistic analytical models can provide an essential 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.8.3.2.7 Performance requirements

A primary objective of these analyses is to ensure that the pile and its penetration are adequate to meet the structure's requirements.

The pile performance for explicit cyclic analyses should be evaluated separately. The pile should have a capacity that provides an adequate margin of reserve above the design actions. In addition, the pile should not settle or pull-out, nor accumulate displacements to the extent that could constitute failure of the structure–foundation system.

A.8.3.3 Overall axial behaviour of piles

No additional guidance is offered.

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 [A.8-3]. 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 [A.8-4].

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 cohesive soils) will not produce the representative axial capacity under ultimate loading conditions.

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* curves

No additional guidance is offered.

A.8.5 Soil reaction for piles under lateral actions

A.8.5.1 General

Generally, under lateral actions, clay soils behave as a plastic material which makes it necessary to relate pile-soil deformation to soil resistance. To facilitate this procedure, 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 pile wall displacement, *y*. By iterative procedures, a compatible set of lateral resistance-displacement values for the pile-soil system can be developed.

For a more detailed study of the construction of *p*–*y* curves, see Reference [A.8-75] for soft clay, References [A.8-76] and [A.8-77] for structured and relatively unstructured stiff clays, Reference [A.8-78] for sand, and Reference [A.8-79] for layered soils.

A.8.5.2 Lateral capacity for soft clay

No additional guidance is offered.

A.8.5.3 Lateral soil resistance–displacement p–y curves for soft clay

No additional guidance is offered.

A.8.5.4 Lateral capacity for stiff clay

No additional guidance is offered.

A.8.5.5 Lateral soil resistance–displacement *p*–*y* curves for stiff clay

No additional guidance is offered.

A.8.5.6 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 cohesive soils, but should be considered for cohesionless soils.

In the absence of project specific data, for an isolated pile a local scour depth equal to 1,5 D and an overburden reduction depth equal to 6 D may be adopted, D being the pile outside diameter; see Figure A.10.

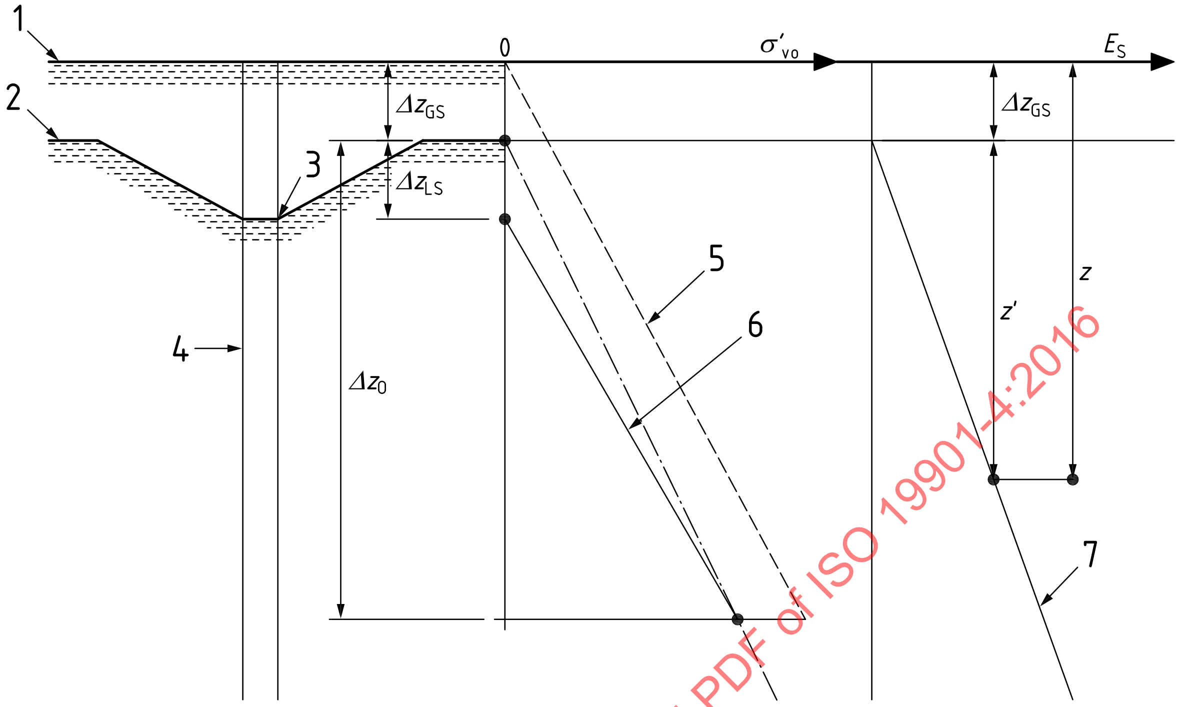
Reduction in lateral soil support is due to two effects:

— a lower ultimate lateral pressure caused by decreased vertical effective stress, $\sigma'_{\mathrm{vo}}(z)$;

— 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.10 suggests one of the methods for evaluating $\sigma'_{\mathrm{vo}}(z)$ and E_{S} as a function of scour depths. In this method, general scour reduces the $\sigma'_{\mathrm{vo}}(z)$ profile uniformly with depth, whereas local scour reduces $\sigma'_{\mathrm{vo}}(z)$ 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[A.10-14] and/or experience, can be used instead.



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 4 for *k*)	z'	final depth below seafloor, after general scour

Figure A.10 — *p-y* lateral support — scour model

A.8.5.7 Lateral soil resistance-displacement *p-y* curves for sand

No additional guidance is provided.

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.[A.8-80] The group resistance is determined by modifying the single pile expressions to account for elastic pile-soil-pile interaction.

The second analysis 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 [A.8-9] 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’.

For more detailed discussions, see References [A.8-80] and [A.8-81].

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.[A.8-80][A.8-81]

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 group analyses. Therefore, multiple analyses should be performed for pile groups using two or more methods of analysis and upper-bound and lower-bound 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.9 Pile installation assessment

A.9.1 General

Drivability studies are carried out in accordance with the principles given in 9.2 and A.9.2 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.[A.9-1]

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.9.2 Drivability studies

References [A.9-2] to [A.9-21] provide relevant information on drivability analyses and the parameters used in these analyses.

A.9.3 Obtaining required pile penetration

No additional guidance is offered.

A.9.4 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);

— 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 [A.9-22] and [A.9-23].

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 [A.9-13], 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.9.5 Driven pile refusal measures

No additional guidance is offered.

A.9.6 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.

A.9.7 Use of hydraulic hammers

No additional guidance is offered.

A.9.8 Drilled and grouted piles

No additional guidance is offered.

A.9.9 Belled piles

In general, drilling of bells for belled piles should employ only reverse circulation methods. Drilling mud should be used where necessary to prevent caving and sloughing. The expander or under-reaming tool used should have a positive indicating device to verify that the tool has opened to the full width required. The shape of the bottom surface of the bell should be concave upward (sides higher than the centre) to facilitate later filling of the bell with tremie concrete.

To aid in concrete placement, longitudinal bars and spiral steel reinforcements should be well spaced. Reinforcing steel can be bundled or grouped to provide larger openings for the flow of concrete. Special care should be taken to prevent undue congestion at the throat between the pile and the bell, where such congestion can trap laitance. Reinforcing steel cages or structural members should extend far enough into the pile for an adequate transfer of forces to be developed.

Concrete should be placed as for tremie concrete, with the concrete being ejected from the lower end of a pipe at the bottom of the bell, always discharging into fresh concrete. Concrete with aggregates of 10 mm (3/8 in) and less in size can be placed by direct pumping. Because of the long drop down along the pile and the possibility of a vacuum forming with subsequent clogging, an air vent should be provided in the pipe near the top of the pile. To start placement of concrete, the pipe should have a steel plate closure with soft rubber gaskets in order to exclude water from the pipe. Care should be taken to prevent unbalanced fluid heads and a sudden discharge of concrete. The pile should be filled to a height above the design concrete level equal to 5 % of the total volume of concrete, placed so as to displace all laitance above the design level. Suitable means should be provided to indicate the level of the concrete in the pile. Concrete placement in the bell and adjoining section of the pile should be as continuous as possible.

A.9.10 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 can be found in ISO 19902:2007, 19.6, which addresses cement grout for connections and repairs.

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 compliance testing can be taken from the returns in addition to the slurry specific gravity measurements. If 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 hours.

A.9.11 Pile installation data

No additional guidance is offered.

A.9.12 Installation of conductors and shallow well drilling

In case of driving pre-curved conductors, special attention should be paid to the stresses generated into the conductor and the structure.[A.9-24]

In soft cohesive soils that are generally encountered in deep water, an alternative technique similar to wash boring, called ‘jetting’, is often used for installing conductors. Insufficient short-term skin friction capacity and excessive settlement can result from a poorly controlled jetting procedure. Additional guidance is provided in References [A.9-25] and [A.9-26].

A.10 Soil-structure interaction for auxiliary subsea structures, risers and flowlines

A.10.1 General

This Clause sets out criteria and recommendations for the design of foundations for subsea structures and for the soil–risers or soil–flowlines interactions as part of a subsea production system. It provides design criteria and methods for soil–risers and soil–flowlines or pipelines interactions that are not addressed in other International Standards from the ISO 13623, ISO 13628 or ISO 19900- series. Other methods, based upon local practice and/or experience, can be used with specialist geotechnical advice and sound engineering judgment.

The following sections address geotechnical design issues for foundations of subsea structures and for the soil–risers or soil–flowlines interactions in soft cohesive soils typical of deep water sites (i.e. with water depth typically >300 m). The design principles outlined in this part of ISO 19901 may be applicable to cohesive soils without any limitation in water depth.

A.10.2 Geotechnical investigation

The scope of the geotechnical investigation should anticipate the requirements for optimal design and code compliance. It should include some combination of sampling and *in situ* testing techniques to penetrations as shallow as 2 m to 3 m for flowlines and steel catenary risers and as deep as 30 m to 40 m for riser towers and top tension risers. Example geotechnical investigation techniques include:

— drilled soil borings with downhole soil sampling and *in situ* testing;

— continuous penetration tests, e.g. cone penetration (CPTU), T-bar or ball tests;

— large-diameter piston drop cores or push samples (core barrel length up to 20 m to 30 m);

— gravity drop cores (core barrel length up to 5 m to 10 m) and box cores (for top 0,5 m of sediments).

Details about equipment and procedures for marine soil investigations are provided in ISO 19901-8.

In addition to identification tests (i.e. bulk density, Atterberg limits, moisture content, grain size distribution, specific gravity, and carbonate content), accurate determination of an appropriate undisturbed undrained shear strength profile is fundamental to the geotechnical design of flowlines and risers. The only laboratory strength tests that can be performed on near-seafloor, very soft clay soil samples are motorized laboratory miniature vane, hand-operated device such as the torvane, and fall cone. In addition, the remolded shear strength should also be measured using the miniature vane and fall cone to evaluate soil sensitivity.

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.

An understanding of *in situ* overconsolidation ratio (OCR) and expected dilatant or contractant behaviour of the soil when sheared can prove useful for the design of flowlines. An understanding of the remolded strength conditions of the soil near the seafloor can also be useful for better understanding the soil response.

Finalization of the site characterization may require the integration of the geotechnical data, geological study and the shallow high-resolution data, depending on the uniformity of geologic units and soil conditions. Such an integrated study can develop maps showing the areal extent of different soil or geologic units and isopach maps showing the depth below seafloor for different soil or seismic horizons and the thickness of different soil or geologic units. The results of the integrated study can be used to assess restraints imposed on flowline and riser design by seafloor features, geohazards, and soil conditions.

A.10.3 Foundations for manifolds and subsea production structures

Foundation configurations that can be utilized include mudmats without or with skirts, driven piles, suction piles, conductors or combinations of these. For well-supporting structures, a foundation system based on support/anchoring on the well conductors housings can also be considered.

As part of the design and selection criteria, it is important to evaluate subsurface obstacles such as boulders or concretions. Possibility of erosion/washout due to drilling should be accounted for in the design, particularly if the distance between the foundation and the well is short and the soil conditions are sensitive to erosion/washout.

The design and installation of shallow foundations with or without skirts is addressed in Clause 7 and A.7. More specific design cases to be considered for the foundations of manifolds or subsea production structures can comprise:

— actions from the tie-in flowlines, spool-pieces, pipelines and umbilicals, with possible coupling between vertical and horizontal actions;

— impact of heat from produced hydrocarbons, particularly if gas hydrates are present;

— installation tolerances and actions possibly due to re-positioning or levelling (if required).

Contingency methods should be established for situations where the foundation fails to penetrate into the seabed. Contingency solutions can be to add weight to assist penetration or to fill grouting into the skirt compartment.

Generally, subsea systems require equipment (templates, manifolds, etc.) to be reasonably level in their final position for proper interface and mating of the various components and subsystems. Depending on the foundation method, levelling of the structure can be achieved by use of jacks or by pumping water in/out of the skirt compartments.

A.10.4 Geotechnical design for steel catenary risers

A.10.4.1 General

The geotechnical properties of the seabed can influence the design conditions for steel catenary risers (SCRs) in two aspects:

— an ultimate limit state associated with excessive bending and tensile stresses in the riser wall; and

— a fatigue limit state associated with cumulative damage to the riser from motion-induced changes in bending stress in the region of the touchdown point.

In an SCR, the maximum static curvature occurs within the suspended part of the catenary and the seabed stiffness has a negligible effect on the maximum curvature. Thus, the seabed properties have essentially no influence on the maximum static in-plane bending stresses within the riser. However, the seabed properties have a significant influence on the shear force in the riser, and hence changes in bending moment due to environmentally-induced motions of the riser. The seabed properties thus affect fatigue calculations.

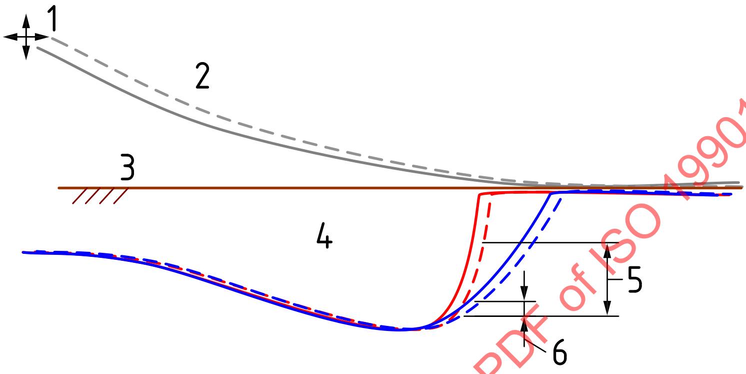
In addition, the seabed properties will affect local out-of-plane curvature of the riser during extreme environmental events or large transverse or out-of-plane motions, particularly where the riser has become partially embedded within the touchdown zone. They can also affect transient bending moments induced during any position changes of the floating facility from which they are suspended.

A.10.4.2 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. Specialist geotechnical advice should be sought to quantify the lateral soil resistance which will usually exceed normal frictional resistance for pipelines lying on the seafloor (see A.10.7). During out-of-plane motion the riser will encounter resistance from the sides of any trench that has formed or from soil berms lying to either side of the riser.

A.10.4.3 Design for fatigue

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. 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. A sketch of the change in maximum pipeline stresses arising from bending moments in the touchdown zone due to example riser motions with both high and low values of soil stiffness is shown in Figure A.11 (Figure A.11 shows stresses, not moments).



Key

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

Figure A.11 — Example stress changes for fatigue calculations[A.10-1]

The cyclic stress range in the touchdown zone depends on the rate of change of the bending moment and, thus, the shear force. Analysis shows that the maximum shear force varies approximately linearly with the logarithm of the soil stiffness. Fatigue laws follow a power law relationship, with damage proportional to a high power (typically about five) of the cyclic stress amplitude.[A.10-2] Even relatively minor differences in the shear force can therefore have a significant effect on the estimated fatigue life, and hence the nonlinear response of the soil needs to be considered.

Either small or large waves can dominate the fatigue damage in the touchdown zone. The majority of fatigue damage can occur from either large waves (not necessarily the most extreme) with low probability of occurrence or continuous motions from small day-to-day waves.

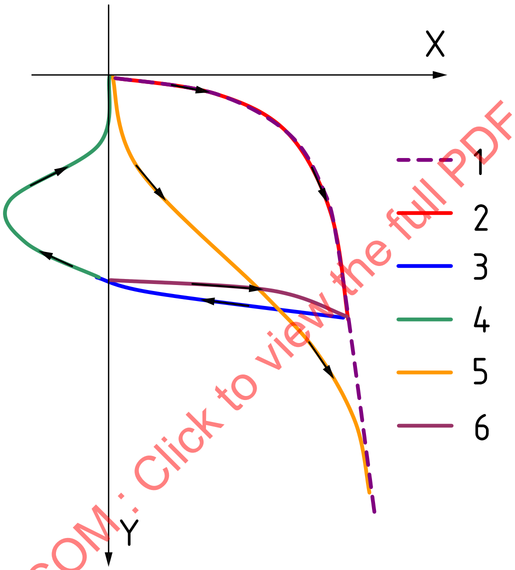
A.10.4.4 Seabed–riser response in vertical plane

A.10.4.4.1 Background

Riser interaction with the seabed involves complex nonlinear processes including plastic penetration during initial touchdown, softening during cycles of upward and downward motion and potential suction-induced tensile resistance prior to breakaway. In most cases, design is undertaken using simplified models where the riser–soil interaction is idealized by a series of linear springs with zero tension capacity distributed along the riser throughout the touchdown zone. Ideally, the choice of spring stiffness should consider the amplitude of vertical displacement and other effects such as the cyclic motion of the riser. While the soil response will also be affected by out-of-plane motion of the riser, the discussion here is restricted to vertical stiffness of the seabed.

Reference [A.10-3] gives the conceptual description of the seabed resistance shown in Figure A.12 for a robust action cycle involving soil–riser separation. Following initial riser penetration into the seabed, unloading occurs as the pipe is uplifted. The soil response in the early stages of uplift is much stiffer than that under conditions of virgin penetration as shown in the 'unloading' curve in Figure A.12. With continued uplift the net resistance force goes into tension ('pipe-soil suction' in Figure A.12) 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.

NOTE The uplift resistance is referred to here as 'suction' although, strictly speaking, under submerged conditions pore pressures normally remain positive. For consistency with much of the published literature, the term 'suction' is retained, understanding that it refers to a net upward force acting on the seabed.



Key

1 backbone penetration curve
2 penetration curve
3 unloading curve
4 pipe-soil suction curve
5 re-penetration curve after breakout
6 cyclic stiffness
X soil resistance, Q (per unit length)
Y penetration, z

Figure A.12 — Conceptual diagram of seabed stiffness[A.10-3]

A.10.4.4.2 Plastic penetration resistance

The backbone penetration curve in Figure A.12 can be estimated by considering the seabed strength profile and an appropriate bearing capacity factor for a given penetration. For conditions where the soil strength profile increases approximately linearly with depth, the limiting penetration resistance per unit length of pipe can be expressed as shown by Formula (A.62):[A.10-4]

$$Q_u = N_c s_u D \approx a\left(\frac{z}{D}\right)^b s_u D \tag{A.62}$$

where

s_u is the undrained shear strength at the pipe invert;

D is the pipe diameter;

z is the depth to the pipe invert;

N_c is a bearing factor defining the backbone curve during virgin penetration;

a is a parameter fitted to results of finite element analyses, with an average value of about 6,0;

b is a parameter fitted to results of finite element analyses, with an average value of about 0,25.

Additional guidance about parameters a and b can be found in Reference [A.10-5].

Allowance for buoyancy effects should also be included.

NOTE s_u refers to an average shear strength (between that measured in triaxial compression, extension and simple shear) or that deduced from a field penetrometer test such as the CPT or the T-bar (see ISO 19901-8).

In certain regions of the world, a crust of higher strength soil exists in the upper 0,5 m to 1 m, before the strength profile reverts to a linear trend.[A.10-6] The potential for the SCR to punch through the crust and the consequences for fatigue studies deserve careful consideration.

A.10.4.4.3 Secant stiffness

The soil resistance behaviour depicted in Figure A.12 can be characterized in terms of equivalent springs having secant stiffness k_v supporting the riser pipe; the secant stiffness, k_v in the vertical plane, is defined by:

$$k_v = \frac{\Delta Q}{\Delta z} \tag{A.63}$$

where

ΔQ is the change in vertical force per unit length of pipe;

Δz is the change in vertical displacement.

The nonlinearity of the riser–soil interaction will lead to a variation in seabed stiffness along the length of the touchdown zone, which can be estimated based on the soil strength profile, $s_u(z)$, and the predicted trench geometry (i.e. trench depth as a function of distance within the touchdown zone). The spatial variation in seabed stiffness is also affected by the temporal variability of the actual point of touchdown.

A hyperbolic model has been proposed for estimating the soil stiffness up to the point at which maximum suction is mobilized.[A.10-3][A.10-7] The model can be expressed as:

$$K = \frac{k_v}{N_c s_u} = \frac{fK_{max}}{f + K_{max} \Delta z / D} \quad \text{(A.64)}$$

where, in addition to previous definitions

K_{max} is the maximum value of the normalized secant stiffness on initial unloading or reloading;

f is the asymptotic value of $\Delta Q/N_c s_u D$ at large displacements.

(i.e. $f = (Q_{initial} - Q_{limit})/N_c s_u D$, where Q_{limit} is Q_{max} for penetration or suction).

NOTE For soft clays, Reference [A.10-3] suggests a value of K_{max} of about 250, which is consistent with the first action cycle of small amplitude laboratory model tests in kaolin.[A.10-8]

A.10.4.4.4 Uplift and breakaway

When the riser pipe is continuously uplifted, a maximum soil uplift resistance is reached, after which uplift resistance declines and breakaway of the pipe from the seabed occurs. The resistance of the soil to uplift can lead to bending stresses in uplift exceeding those in lay-down. In contrast, separation of the pipe from the soil tends to relieve bending stresses in the pipe. Accordingly, realistic estimates of the magnitude of maximum suction resistance and the displacement levels associated with suction mobilization and when breakaway occurs are important to characterize soil–riser interaction accurately.

Soil resistance to uplift of the riser pipe can arise from two mechanisms. The first is the ability of the soil to resist suction. The second is the resistance mobilized by backfill of soil above the pipe created by processes such as deposition or collapse of the side walls of the trench.

The maximum suction force per unit length of pipe, $Q_{s\,max}$, depends on a number of factors such as the effects of cyclic movement, the pipe velocity and the time over which the uplift resistance is sustained. These can be expressed through factors applied to the (compression) bearing capacity as shown in Formula (A.65).[A.10-3]

$$Q_{smax} = f_c f_V f_t N_c s_u D \quad \text{(A.65)}$$

where

f_c is a dimensionless cyclic factor;

f_V is a dimensionless velocity factor;

f_t is a dimensionless time factor.

Recommendations for factors f_c, f_V, and f_t are given in Reference [A.10-3].

For conditions of cyclic and fatigue loading, Reference [A.10-3] recommends the use of a remoulded strength s_{ur} rather than intact strength s_u, in addition to a cyclic reduction factor f_c, although it can be more consistent to use the original intact shear strength as the benchmark, relying on the various factors to quantify adjustments in estimating $Q_{s\,max}$.

Video surveys have shown that risers often cut a trench of significant depth in the seabed, which is considered further in A.10.4.5. Additional uplift resistance will occur where partial backfilling of the trench occurs, leaving the pipe embedded. The trench backfill is likely to be a product of mixing with water as well as remoulding; therefore, its strength is likely to be less than the remoulded strength of the seabed soil.

For conditions of no trench backfill with uplift resistance being mobilized purely from suction, Reference [A.10-3] proposed the relationship shown by Formula (A.66) for break-out displacement

Δz_b, i.e. the uplift displacement at which the pipe completely detaches from the soil, measured from the point at which the net force, Q, becomes negative.

$$\Delta z_b = \zeta_V \zeta_t D \quad \text{(A.66)}$$

where

ζ_V is a dimensionless velocity factor;

ζ_t is a dimensionless time factor.

Reference [A.10-3] provides recommendations for the factors ζ_V and ζ_t based on STRIDE and CARISMA data.

For the case of a backfilled trench, general relationships analogous to Formula (A.66) have not appeared in the published literature. However, relationships for uplift–displacement behaviour have been developed on an ad hoc basis for specific sites (e.g. Reference [A.10-9]).

A.10.4.4.5 Stiffness adjustment for cyclic loading

Cyclic loading is recognized to degrade the soil stiffness. Based on cyclic model pile-soil tests in kaolin, Reference [A.10-8] reported values of normalized cyclic stiffness, K, of less than 5, in contrast to monotonic K values ranging from around 250 down to 40 at $\Delta z/D = 0{,}025$. Model tests also suggested that the stiffness reduces by a factor of 10 to 20 where soil–riser separation and re-contact occurs.[A.10-10]

The magnitude of the distance along the pipe over which soil–riser separation occurs will vary with water depth, riser properties, type of floating facility and environmental conditions. Table A.4 summarizes typical motions of the riser in the touchdown region for different storm conditions for Spar platforms in the Gulf of Mexico. For typical riser diameters of 0,3 m to 0,4 m, these distances correspond to about ±15 m for day-to-day wave loading and ±25 m for extreme storm conditions. Transverse or out-of-plane vessel motions could lead to the trebling of the range of separation.

Table A.4 — Summary of distance and occurrence of SCR TDP motions[A.10-9]

Motion	Probability of occurrence	Limit of in-plane TDP motions	Limit of transverse TDP motions
Day-to-day	95 %	±43 D	±0,5 D
Extreme storm	2,5 %	±70 D	±1 D
Second-order vessel motions	2,5 %	±200 D – 260 D	±7 D

On the catenary side of the mean TDP, the relevant soil stiffness can be taken as that associated with large displacements, and hence an order of magnitude lower than the maximum unload–reload stiffness. On the flowline side of the TDP, the gradual decay in magnitude and frequency of the motions suggest that, in the zone between day-to-day and extreme storm conditions in Table A.4, the ‘average’ operative value of stiffness should be increased in steps towards the maximum value. Histograms of the vertical SCR motion can assist in assessing the magnitude and spatial variation in stiffness decay.

In practice, software used for assessing fatigue damage can be limited in the extent to which a spatially varying seabed stiffness can be implemented. An equivalent effect can be achieved with a suitable nonlinear model for riser–seabed interaction incorporating a relatively soft backbone curve and higher unload–reload stiffness, as indicated in Figure A.12. Such nonlinear models are now starting to become available in commercial software packages for riser design.

A.10.4.5 Self-trenching

A.10.4.5.1 Process and features

Riser movements at the touchdown zone produce seafloor trenching that affects stress analysis and fatigue performance of the riser. Trenching is a process of soil scour and ploughing due to riser responses to global motions of the floating system. The process generally takes a duration of months to years of cumulative soil displacements to form trenches, but can be accelerated with greater incidence of environmental events, heave-prone vessels or vortex-induced vibration.

Observations of riser trenches reported by Reference [A.10-11] reveal a typical, ladle-shaped profile that tends to be deepest near the nominal touchdown point and gradually slopes up to the natural seafloor towards the flowline. The riser is often buried inside the trench in the region of the mean touchdown point. In plan, trenches are generally shaped as a bell mouth, with the widest portion on the vessel side of the touchdown and progressively narrowing towards the flowline. A generalized trench shape is provided in Figure A.13.

Trench dimensions for a specific riser depends on several factors, including:

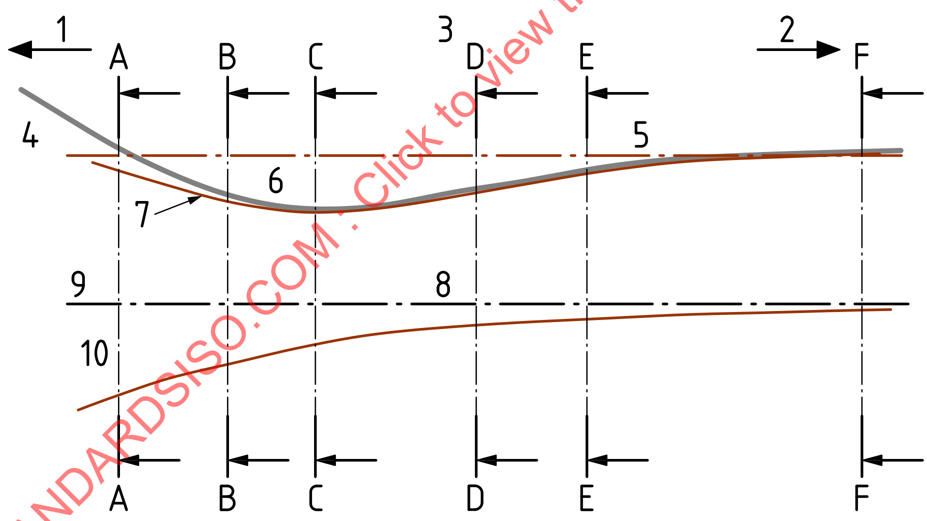
— soil conditions;

— riser dimensions and configuration (e.g. pipe-in-pipe);

— environmental conditions;

— time after installation;

— vessel type.



Key

1	vessel	6	ladle shaped trench profile
2	flowline	7	approx. riser touchdown point (TDP)
3	profile view	8	plan view
4	riser	9	centre line
5	seafloor	10	bell mouth shaped trench profile

Figure A.13 — Overview of generalized trench[A.10-11]

A.10.4.5.2 Influence on analysis

The impact of a trenched seafloor on SCR touchdown region fatigue performance is not well defined due to competing factors related to trench geometry and soil conditions. In contrast, trenching can cause large bending stresses in the riser for transverse motions at or near the touchdown. The riser can be overstressed in cases of abrupt interaction with a trench during extreme environmental events or large transverse or out-of-plane motions. Also, persistent transverse motions inside a trench by prevailing environmental conditions, second-order motions and vortex-induced vibration can adversely impact the fatigue life at or near the riser touchdown.

A.10.4.5.3 Three-dimensional motion

Under extreme conditions, such as for broken mooring lines, transverse or out-of-plane motions of the vessel or platform can lead to three-dimensional motion of the riser sufficient to cause it to 'break-out' from the current trench. Consideration should therefore be given to wide sweeps of the nominal SCR touchdown point as the surface vessel translates through large horizontal distances, which can be in the range 5 % to 10 % of the water depth. This distance is likely to be larger than the largest trench width considered previously. The surface vessel can swing back and drag the riser large distances past the original SCR trench, before reaching equilibrium at a position that is not quantifiable in a failure scenario like this.

Under normal (i.e. with intact moorings) operations in major storms and large current events, it is also possible for the floating support structure to lift an SCR out of its trench and set it down in virgin soil many trench widths away as the surface and subsea environmental conditions build up, possibly change direction and then ease off.

In either of the above situations, a new trench can form at the new location, which will affect the fatigue life of the riser. A more critical issue, however, is that the break-out resistance from the original trench can result in high localized bending moments and a potential ultimate limit state. The effect of the high break-out resistance during three-dimensional motion of a self-trenched riser should be quantified.

A.10.5 Geotechnical design for top tension risers

A.10.5.1 General

The design of top tension risers near or below the blow-out preventer (BOP) stack is dependent on the riser (conductor)–soil interaction. The following two design issues should be considered:

a) the reaction of the riser at the ultimate limit state when the host facility has moved a considerable distance from the mean position;

b) the fatigue that occurs within the riser as a result of repeated cyclic motions with a range of amplitudes and frequencies.

The first problem occurs during a move-off condition or during environmental loading. The move-off case can be an intentional static type of condition that occurs when the host facility is moved to facilitate drilling operations or when the host temporarily loses stationkeeping capabilities.

The fatigue problem is governed by cyclic actions that occur throughout the life cycle of the riser–conductor system. These actions can occur from:

— environmental wave, wind and current actions on the host facility;

— vortex induced vibrations (VIV) on the host facility or riser;

— environmental actions (wave and current) mostly in the top portion of the riser.

Analyses have shown that peak actions are not the major contributor to the fatigue damage. Rather, smaller more frequent actions are responsible for most fatigue damage. Therefore, characterization of the soil response at small amplitude displacements is particularly important for the fatigue problem. Previous work indicates that the criteria in ISO 19902 can underestimate the soils lateral stiffness, especially at low amplitude displacements. It is often difficult to assess whether soft or stiffer soil stiffness estimates are the more conservative. Stiffer estimates suggest that the critical cyclic bending moments will occur above the seafloor while softer estimates would suggest the opposite.

Finally, temperature effects will also have an impact on the conductor. Increased temperature can result in

— upheaval forces on the conductor,

— changes in the shear strength of the soil around the conductor, or

— dissociation of seafloor hydrates.

The impact of temperature changes on the soil properties along the conductor is likely to be more severe for axial actions. However, the axial capacity is quite robust due to the length of the conductor and associated casing strings. The impact on lateral actions is mitigated due to the constant temperature condition imposed by seawater at the seafloor.

A.10.5.2 Soil response

The response of the near seafloor portion of top tension riser–conductors to fatigue loading is highly dependent on soils about 15 m to 20 m below the seafloor. The overall challenge of assessing this lateral response is analogous to that of a laterally loaded pile. Accordingly, lateral soil springs provided for offshore piles have often been used to assess the lateral response of a conductor. Despite the apparent similarities, the following differences exist.

— The soil springs for piles were originally developed for steel jackets subjected to large storm actions. As such, the primary intent was to characterize the soil near yield while less attention was paid to the soil response at smaller displacements.

— The maximum moments developed in a pile are relatively insensitive to the lateral soil response. Differences in the soil springs will tend to change the location rather than the magnitude of the maximum moment. Since the wall thickness for offshore piles is usually constant, inaccuracies in the p–y springs will have a lesser impact. Top tension risers–conductors, however, are assembled in sections with connectors. The connectors are critical locations for fatigue.[A.10-47]

— The recommendations from 8.5 and A.8.5 were based on tests with loading applied over a few days while the loading that cause fatigue are applied over a much shorter time period.

— The relationship between cyclic bending moment and fatigue life is highly nonlinear so that with many cycles fatigue life is highly sensitive to soil stiffness.

A.10.5.3 Development of *p–y* springs via finite element analysis

Instead of relying exclusively on the method described in 8.5 for lateral soil resistance–displacement analyses of riser–conductor systems, the *p–y* springs can alternatively be developed using the finite element (FE) method. This subclause provides guidelines for this approach.

An important aspect of developing *p–y* springs with the finite element method is developing a representative soil model for riser–conductor problems. The small strain or initial shear modulus is an important part of this soil model. The initial shear modulus, G_{max}, can be determined from resonant column testing on samples taken during the soil investigation (see ISO 19901-8). Samples should be taken from depths where the maximum fatigue damage might occur, for example the top 15 m of soil. The resonant column test results should be adjusted for actual site conditions, for example using the correlations proposed in Reference [A.10-12].

Values of G_{max} obtained from resonant column tests should also be modified to account for:

— the increase in shear modulus that occurs after primary consolidation;

— the reduction in shear modulus due to the lower rate of loading expected for riser–conductor types of loading, compared to the higher rate in the resonant column tests; and

— cyclic degradation, with the influence on steel stress and fatigue behaviour being bracketed between pre- and post-cyclic behaviour of the soil mass.

The increase per log cycle in time due to secondary consolidation has been estimated as proportional to the square root of the soil plasticity index.[A.10-12] This increase will be offset in part by the reduction due to the lower rate of loading which, in absence of site-specific data, can be approximated as a 10 % reduction in shear modulus per log difference in frequency relative to the resonant column tests.

Alternatively, Reference [A.10-13] presents the following for a normalized shear modulus (G_{max}/s_{uDSS}) for normally consolidated clays:

$$\frac{G_{max}}{s_{uDSS}} \approx \frac{300}{I_P / 100} \qquad \text{(A.67)}$$

where

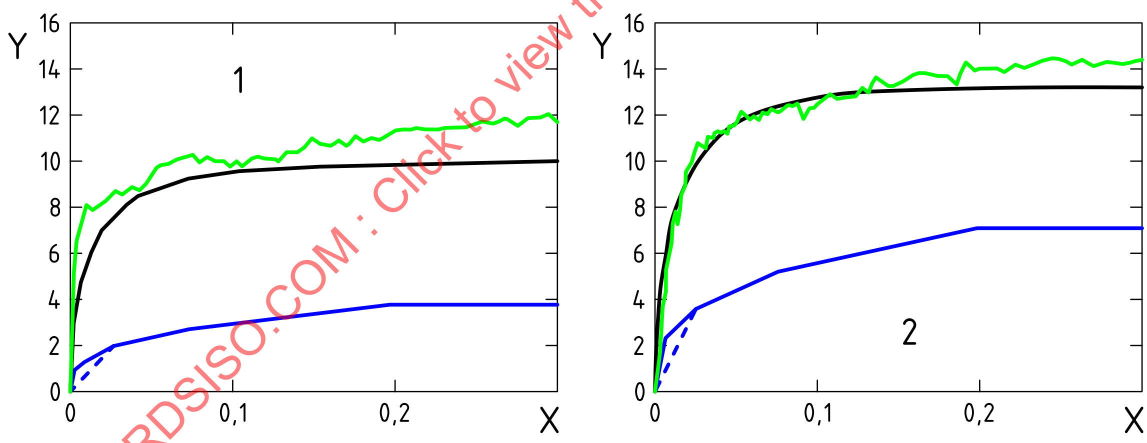
s_{uDSS} is the undrained shear strength from direct simple shear tests;

I_P is the soil plasticity index.

Generally, values of normalized shear modulus for overconsolidated clays are lower than those for the normally consolidated data.[A.10-13]

Figure A.14 shows example *p*–*y* curves developed with the finite element approach compared with 8.5.3.[A.10-14] Also shown in the figure are centrifuge results for tests where the action was applied in less than one minute. The green and black curves show the direct comparison between the centrifuge results and finite element model. The blue curve represents 8.5.3 recommendations for soft clay.

Additional details for developing *p*–*y* curves through numerical modelling procedures are described in Reference [A.10-15].



Key

1 depth = 1,4m = 1,5 diameters
2 depth = 7,3m = 8 diameters
X lateral displacement/diameter, y/D
Y unit pressure/shear strength, p/s_u

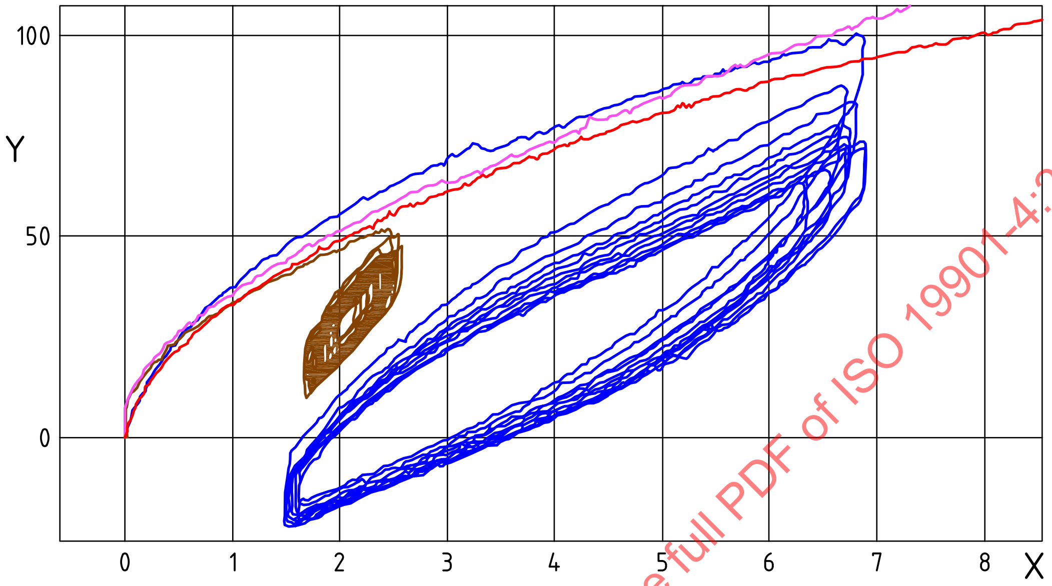
dashed blue: API RP2A[2] (2000)
solid blue: API RP2A Addendum #3 (2008)
black: FE analyses
green: centrifuge tests

Figure A.14 — Comparison between *p*–*y* springs measured in centrifuge tests on kaolinite and computed using the finite element method with 8.5.3[A.10-14]

A.10.5.4 Additional considerations

In addition to soil stiffness at small strains, the soil should also include effects of work hardening. This work hardening results from either elasto-plastic isotropic hardening of the soil at larger strains, or kinematic hardening from cyclic loading.

Figure A.15[A.10-14] presents results that demonstrate this potential impact. This figure shows that cyclic loading can degrade the soil stiffness. If the static *p*–*y* backbone curve is used in the analysis at the mid-point for the cyclic action, the actual unload–reload secant stiffness (as is used in several riser fatigue software products) is greater. Therefore, using the tangent stiffness from the backbone curve can produce conservative results (more fatigue damage) when the critical fatigue point is below the seafloor. However, if the critical fatigue point is above the seafloor, softer soil springs can result in a non-conservative solution.



Key

X prototype pile head displacement (in, 1 in = 0,0254 m)

Y prototype pile head load (kips, 1 kip = 4,448 kN)

test 2 – final monotonic push

backbone curve – test 1

test 2 – 10 cycles

test 2 – 50 cycles

Figure A.15 — Expanded view of pile head displacement versus lateral pile head loading[A.10-14]

A.10.5.5 Summary and recommendations for top tension risers

A summary and recommendations for top tension risers are:

a) Top tension riser–conductor design should consider both ultimate and fatigue limit state. Because of variable wall thickness and irregular geometries, the critical bending moment could occur either above or below the seafloor.

b) The *p*–*y* curves specified in 8.5.3 for piles provide a significantly softer response than *p*–*y* curves developed via finite element analyses. Advanced soil models should be used based on site-specific soils data that captures both small displacement soil behaviour and work hardening from cyclic loading.

c) Although finite element modelling produces a stiffer soil response, model tests demonstrate cyclic softening with repeated loading cycles.

d) Whenever the critical fatigue point is below the seafloor, the tangent stiffness from adjusted static *p*–*y* curves based on finite element analyses can be conservative because the tangent stiffness at the mid-point of cycling can be less than the unload–reload stiffness. If the critical fatigue point is above the seafloor the *p*–*y* curves obtained from 8.5.3 for soft clay can be non-conservative. For this case the amount of cyclic degradation should be conservatively estimated, i.e. less degradation.

e) Developing *p*–*y* curves with finite element analysis requires expert skills and is time consuming. If the critical bending moments are below the seafloor, the curves obtained from 8.5.3 for soft clay can be initially used. If this initial attempt leads to unacceptable failure levels, then the site-specific data can be used to determine the benefit from increasing soil stiffness.

f) For a drilling riser with heavier lower stacks (BOP), the soft soil (low soil stiffness) can cause the system natural frequency to move into wave energy zone. So, using stiffer soil (or fixed at seafloor) does not guarantee the results to be in the conservative side even if the critical fatigue point is above the seafloor.

A.10.6 Foundation design for riser towers

A.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 connection to the surface vessel or platform is generally ensured by flexible jumpers. A riser tower supports axial tension generated by buoyancy and by cyclic wave action, and should be secured into the seabed. A significant part of the tension acts permanently during the life of the development.

There are a number of possible foundation options: gravity base, suction piles and driven piles. The concept selection for the foundation should take into account the soil data, the installation aspects, as well as the in-place performance. A preliminary conceptual study should be performed to select the most appropriate foundation type.

A.10.6.2 Actions and resistance factors

A.10.6.2.1 Actions

Design actions should be evaluated for the following conditions:

— foundation installation and retrieval;

— normal operating conditions;

— extreme conditions.

Action combinations should be selected so as to anticipate the most unfavourable result, for each of the stability mechanisms and deformation analyses performed.

A.10.6.2.2 Recommended resistance factors

Resistance factors should be as recommended by API RP 2T[5] for driven piles and by ISO 19901-7 (and A.11.5.2) for other anchor types such as suction piles.

NOTE 1 The ISO 19901-7 resistance factors were developed without considering the permanent uplift actions on suction piles. Reference [A.10-16] provides information on the potential uplift response under sustained actions.

API RP 2T[5] does not address the case of gravity actions explicitly. Also, Reference [5] 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 equation”.

A.10.6.3 Soil design parameters

High quality *in situ* measurements and/or soil borings should be obtained to select the soil design parameters. It is critical to establish whether any high permeability layers are present within or above the zone of influence for reverse end bearing under sustained loading.

The depth of the geotechnical borings should exceed the foundation depth by at least three anchors diameters unless a more regional site characterization has shown that there are no major changes in stratigraphy within that depth. If there are no major changes in stratigraphy, the depth beneath the anchor tip can be reduced to one anchor diameter. Input of a geotechnical specialist is recommended if the boring depth is reduced.

The number of geotechnical borings should be defined as a function of the soil variability. One boring should be performed at each anchor location when unusually large lateral variability of the soil properties is expected. The main soil properties and design parameters that are needed are defined in ISO 19901-8.

A.10.6.4 Design issues

A.10.6.4.1 General principles

The following are the general principles that should be considered in assessing the stability of riser tower foundations.

— Limit equilibrium methods can generally be used to evaluate the capacity of riser tower foundations. The shear strength used in the analysis should account for the effects of creep and potential drainage under sustained action and cyclic degradation. The reduction in effective stresses and shear strength due to potential drainage can be studied by finite element analyses.

— Due consideration should be incorporated into the design regarding displacement and deformation during the life of the foundation. Where displacement and deformation are critical, complex analysis methods can be warranted. The displacement analysis should include contributions from undrained shear strains due to application of the sustained action, undrained creep during the sustained action, and permanent and cyclic components from the wave loading. Displacements due to shear strain, volumetric strains and flow of water through the soil due to potential drainage during the sustained action should also be considered.

— The anchors should be installed within specified tolerances of tilt and mis-orientation. The design analyses should account for the effect of the tolerance limits.

— Installation should be planned so as to ensure the foundation can be properly seated at the intended site without excessive disturbance to the supporting soil. Where excessive disturbance does occur, this should be considered in the assessment of foundation capacity.

— Measures should be taken to avoid erosion and scour of the soil beneath or near the foundation base.

Where removal is anticipated, an analysis should be made of the forces generated during removal to ensure that removal can be accomplished with the means available.

A.10.6.4.2 Geotechnical design methodology for the foundation

The design of driven piles and gravity base is covered by the methods described in this part of ISO 19901 and by the recommendations from Reference [5].

The design of suction caissons is covered by the recommendations in A.11.5.2, with more detailed aspects considered in the literature (e.g. References [A.10-16] to [A.10-26] and Reference [A.11-22]). The following aspects should be considered:

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 action and for the sum of permanent and cyclic actions. If the cyclic actions (quick/short-term) are small compared to the permanent actions (sustained/long term), the permanent action condition can be critical because the strength can be smaller for this condition than for the condition where the cyclic actions are included. The following should be considered.

— The undrained shear strength for the permanent action condition should be reduced to account for creep effects (e.g. Reference [A.10-27]). The effect of pore pressure redistribution and swelling should also be considered since this can lead to reduction in effective stresses and undrained shear strengths, and hence reduce the capacity under transient wave loading.[A.10-19] In cases where the permanent/sustained action lasts for months, it is also necessary to consider whether drained conditions might develop, and the extent to which full base suction (reverse end bearing) can be maintained. The possibility of drainage channels between inside skirt stiffeners, above inside stiffeners and extended skirt wall thicknesses, and along open cracks outside the anchor at the active side of the anchor should be considered. References [A.10-16] and [A.10-28] show that the capacity of a typical suction caisson under sustained loading can be only 70 % of the capacity for rapid loading.

— Drainage and pore pressure redistribution can also influence the undrained shear strength under cyclic loading, but the shear strength for the sum of permanent and cyclic actions can be higher than for the permanent action due to rate effects.[A.10-27]

The potential of gapping 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 (e.g. References [A.10-18]). If gapping or trapped water is possible, the effect that such gaps can have on the drainage path and on resistance due to lack of contact along the wall should be considered.

The capacity of suction caissons can depend on passive under-pressure (i.e. reverse end bearing, REB) inside the caisson. If passive under-pressure is relied upon, proper sealing can be critical, especially for the part of the under-pressure generated by long period environmental actions such as loop currents. The anchor top can be sealed, but if the valve seals cannot be guaranteed, consideration should be given to either a back-up cap behind the valves or a monitoring program to ensure the desired integrity over the lifetime of the suction caissons. If proper sealing is not ensured, the suction caissons should be designed to resist sustained uplift action without taking passive under-pressure into account.

A.10.6.5 Inspection and monitoring

An inspection program should be considered as integral part of the foundation design. The inspection program should include the use of instrumentation to monitor critical aspects of the foundation performance during both installation and operation.

If at any time during the service life of the structure, the inspection program reveals conditions or behaviour which are detrimental to the integrity of the foundation or structure, then maintenance or remedial measures should be carried out as necessary.

A.10.7 Geotechnical design for flowlines and pipelines

A.10.7.1 General

Pipeline design should consider ultimate and fatigue limit states related to the stresses in the pipeline and the movements of the associated end connections including sections which transition into a catenary riser. The response is influenced by the geotechnical interaction forces between the pipeline and the seabed as well as other external actions on the pipeline and internal actions within the pipeline.

Geotechnical advice should be sought to predict the as-laid pipeline embedment and the resulting force-displacement response in the axial and lateral directions. The most basic forms of axial and lateral pipe–soil model are linear elastic – perfectly plastic. More sophisticated designs require more complex pipe–soil models. Either upper or lower bound values of the pipe–soil interaction forces can be critical for a limit state. Each bound should be assessed as necessary.

A.10.7.2 Actions on pipelines

The actions and motions imposed during laying govern the pipeline embedment and any residual tension at the start of operation and initial in-service performance. Hydrodynamic loading and subsequent scour and seabed liquefaction processes can lead to changes in embedment during the operational life of the pipeline.

After installation, the actions imposed on an individual element of pipeline should be balanced by reaction forces from the soil. The actions on the pipeline element arise from adjacent or bounding elements of the pipeline, a connected steel catenary riser, hydrodynamic or thermal loading, and internal and external pressure. The compressive axial force created by operating cycles of internal pressure and temperature can lead to lateral buckling of the pipeline or the accumulation of axial movements (pipeline 'walking').

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.10.7.3 Soil reaction forces

A.10.7.3.1 Pipeline–soil interaction models

The interaction between the pipeline and the seabed is incorporated into the structural analysis of a pipeline by attaching pipe–soil model elements at intervals along the pipeline. This approach is analogous to the t–z and p–y action transfer methods of analysing pile response.

For some simple pipeline design functions, the pipe–soil response is represented by limiting values of axial or lateral pipe–soil resistance or bi-linear elastic – perfectly plastic behaviour in the axial and lateral directions. The pipe–soil resistance is typically expressed as an equivalent friction factor, linking the limiting resistance to the effective pipeline weight. However, the axial and lateral resistances can depend on factors other than the pipeline weight, in particular the embedment. Therefore, a friction factor is not an intrinsic soil property.

To capture the more complex effects of interaction, particularly the large displacement behaviour, it is necessary to model other aspects of the response, including brittle break-out behaviour and cyclic berm growth during lateral movement.[A.10-29]

A.10.7.3.2 Drained and undrained soil behaviour

In fine-grained cohesive sediments, pipeline laying is usually an undrained process. Dissipation of the lay-induced excess pore pressure typically takes days or weeks. Lateral pipeline movements generally involve undrained deformation although consolidation between events can cause disturbed soil to regain strength. Axial pipeline movements can be drained or undrained since the relevant drainage distances are shorter than for lateral movement.

In coarse-grained cohesionless sediments, pipeline installation and operation will generally occur under fully drained conditions. In a design analysis, the anticipated rates of axial and lateral pipeline movement should be compared with the relevant rates of soil drainage and consolidation to establish whether drained or undrained conditions will prevail.

A.10.7.4 Analysis of pipeline–soil interaction

A.10.7.4.1 Vertical penetration

A.10.7.4.1.1 Lay effects

Observations show that the as-laid pipeline embedment is typically much greater than would be expected from the static weight alone, due to over-stressing and dynamic motion in the touchdown zone during laying.[A.10-30] The contact bearing stresses (or vertical force per unit length) between the pipeline and the soil in the vicinity of the touchdown point exceeds the as-laid self-weight of the pipeline due to the catenary shape.

The degree of over-stress is dominated by the bending rigidity of the pipeline, the apparent stiffness of the seafloor, and the tension in the pipeline in the vicinity of the touchdown point.[A.10-31] The applicable apparent stiffness of the seafloor is a secant stiffness for the anticipated degree of plastic penetration and could be much lower than that customarily used for fatigue assessment within the touchdown zone of a steel catenary riser. In deep water, the over-stress can be negligible because of soft seafloor conditions. If the pipeline is laid empty then the maximum static loading can occur during hydrotesting, when the pipeline is heavier.[A.10-31] The as-laid embedment can therefore be governed either by the as-laid weight (with a touchdown overstress and dynamic lay effects) or by the hydrotest condition.

Vessel motion, changes in pipeline tension and hydrodynamic loading of the hanging pipe will induce a combination of vertical and horizontal motion of the pipeline at the seafloor during laying References [A.10-30], [A.10-32], [A.10-33] and [A.10-34]. Even small lateral or vertical movements can cause disturbance, local softening and erosion of the seafloor in the touchdown zone, increasing the pipeline embedment.

A.10.7.4.1.2 Static vertical penetration response

For seabed sediments where drained conditions will prevail, conventional bearing capacity approaches can be used to estimate the static pipeline penetration. The pipeline can be treated as a surface strip foundation of width equal to the (nominal) chord length of pipe–soil contact at the measured embedment. In most cases, however, the penetration resistance will be such that minimal embedment of the pipeline will be predicted based on static loading. Other processes such as cyclic motion of the pipeline, scour and partial liquefaction of the seabed will determine the as-laid embedment depth.

For fine-grained cohesive sediments, where undrained conditions prevail during embedment, theoretical solutions for estimating the pipe penetration resistance have been provided in References [A.10-4], [A.10-35] and [A.10-36]. These solutions use the conventional bearing capacity equation, modified for the curved shape of a pipeline. In soft soils, the enhanced soil buoyancy created by heave can be significant.[A.10-5]

A.10.7.4.2 Axial soil resistance

Axial pipeline movement involves shear failure at or close to the pipeline–soil interface. The vertical effective contact force can be used to calculate the effective stresses and forces at the pipeline–soil interface. The integrated normal contact stresses around the pipeline periphery exceed the vertical contact force due to the curved shape of the pipe surface.[A.10-31] Using the enhancement factor, ζ, to account for this effect, the drained axial resistance per unit length of pipeline, T, is given by:

$$T = \mu N = \mu \zeta V \tag{A.68}$$

where

μ is the pipeline–soil friction coefficient, which can be alternatively expressed in terms of an interface friction angle δ, where $\mu = \tan \delta$;

ζ is the enhancement factor;

N is the integrated normal contact force;

V is the current vertical force (essentially pipeline weight).

Based on an elastic solution, Reference [A.10-31] provides the following expression for ζ :

$$\zeta = \frac{N}{V} = \frac{2\sin\theta_{D'}}{\theta_{D'} + \left(\sin\theta_{D'}\cos\theta_{D'}\right)} \tag{A.69}$$

where

$\theta_{D'}$ is the half-angle of the pipeline–soil contact perimeter, which varies with normalized embedment z/D, in accordance with:

$$\cos\theta_{D'} = 1 - \frac{2z}{D} \tag{A.70}$$

Due to the stress-dependency of soil friction angle, this parameter should be assessed by tests conducted at the correct stress level.[A.10-31][A.10-37] During undrained axial pipeline movement, the apparent friction coefficient can increase or decrease depending on whether negative or positive excess pore pressure is generated by shearing at the interface.

Laboratory tests can be used to assess an appropriate friction coefficient, μ, for drained shearing including separate values for peak and residual resistance. Tests can be conducted in a low stress shear box or on a tilt table, using a sample of the pipeline surface coating. Alternatively, a model pipe section can be tested in a larger test chamber. These tests should replicate the relevant lay-induced consolidation history and the speeds and pause periods relevant to the design situation. It should be established whether undrained or partially drained conditions can apply during pipeline motion. These conditions can lead to a significant reduction in the apparent friction coefficient, μ, compared to the drained case.

A.10.7.4.3 Lateral soil resistance

Lateral pipe–soil resistance during break-out and large amplitude cyclic movement is influenced by the initial pipe embedment and weight, the development of soil berms ahead of the laterally sweeping steel catenary riser or pipeline segment, and the soil properties. Two characteristic types of large amplitude lateral response are typically observed (Figure A.16) depending on the ratio of the pipeline weight to the seabed strength, V/s_uD, i.e.

— For values of V/s_uD below about two ('light' pipe), the pipeline tends to rise after breaking out from the as-laid position. As the pipe rises, the lateral resistance reduces from the break-out value to a residual resistance. The pipeline sweeps horizontally with a berm of soil being pushed ahead of the pipe. This mechanism governs the residual resistance, H_{res}. Subsequent cycles of lateral movement lead to a steady increase in the restraint provided by the soil berms [Figure A.16 a)]. The 'light' pipe form of response is also typically observed in drained conditions.

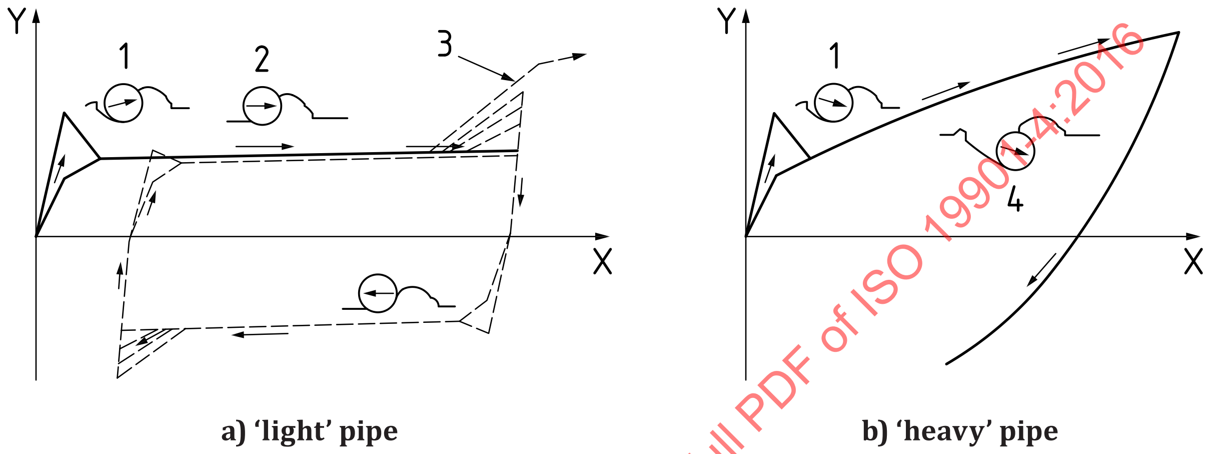
— For values of V/s_uD greater than about two ('heavy' pipe), the pipeline typically moves downwards after the initial break-out resistance is mobilized. This downward movement, coupled with the growth of a soil berm ahead of the pipe, leads to steady increase in the lateral resistance [Figure A.16 b)].

Empirical expressions exist for predicting lateral pipeline–soil resistance, which have evolved primarily through calibration against model tests.[A.10-38][A.10-39][A.10-40][A.10-41] These expressions are subject to significant uncertainty and their relevance should be established for a particular design situation. In undrained conditions, the break-out resistance, H_{brk}, is generally divided into two contributions:

a) a component proportional to the current vertical action, V (which is essentially the pipeline weight);

b) a passive resistance component linked to the embedment depth of the pipeline, z.

An alternative approach is to use yield envelopes (or interaction diagrams) in vertical and horizontal action space that bound the allowable action combinations for a given pipeline embedment.[A.10-36][A.10-42]



Key

1 breakout resistance, H_{brk}
2 residual resistance, H_{res}
3 berm resistance, H_{berm}
4 accumulating passive resistance
X horizontal displacement, u
Y horizontal resistance, H

Figure A.16 — Types of large amplitude cyclic lateral pipe–soil response in undrained conditions

For assessment of fatigue performance of a pipeline during large amplitude cyclic movement, the size and strength of the growing berm formed ahead of the sweeping pipeline segments and at the extremities of the pipeline movement are significant.[A.10-29]

Expressions for assessing lateral pipeline–soil resistance at break-out and during cyclic motion can be calibrated and validated by laboratory models, centrifuge tests,[A.10-43][A.10-44] seabed test sections[A.10-45][A.10-46] or analytical methods.

Special care should be taken where the local effective flowline weight (i.e. localized touchdown stress) changes, e.g. at touchdown points around sleepers, natural seafloor spans, and at the start of distributed buoyancy sections. Location-specific lateral resistance parameters might be required in these zones.

A.11 Design of anchors for floating structures

A.11.1 General

This Clause provides recommendations for the 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.

The options that are available for anchoring floating structures include:

— drag embedment anchors;

— anchor piles, comprising driven, suction, vibro-driven, jetted, or drilled and grouted piles;

— plate anchors, including drag-embedded, or direct-embedded plate anchors;

— other anchor types, such as gravity anchors and gravity-embedded anchors (free-falling 'torpedoes').

Recommended design criteria and ultimate limit state (ULS) design safety factors for anchoring systems are given in ISO 19901-7. In selecting anchor options, consideration should be given to soil conditions, required system performance and reliability, installation and the test loading (where relevant). 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 Annex to the extent currently possible. General statements are also used to indicate that considerations should be given to some particular aspects, 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 that a high quality, high-resolution geophysical survey be performed 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, CPTU) and to aid in the interpretation of the acquired geotechnical data.

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.

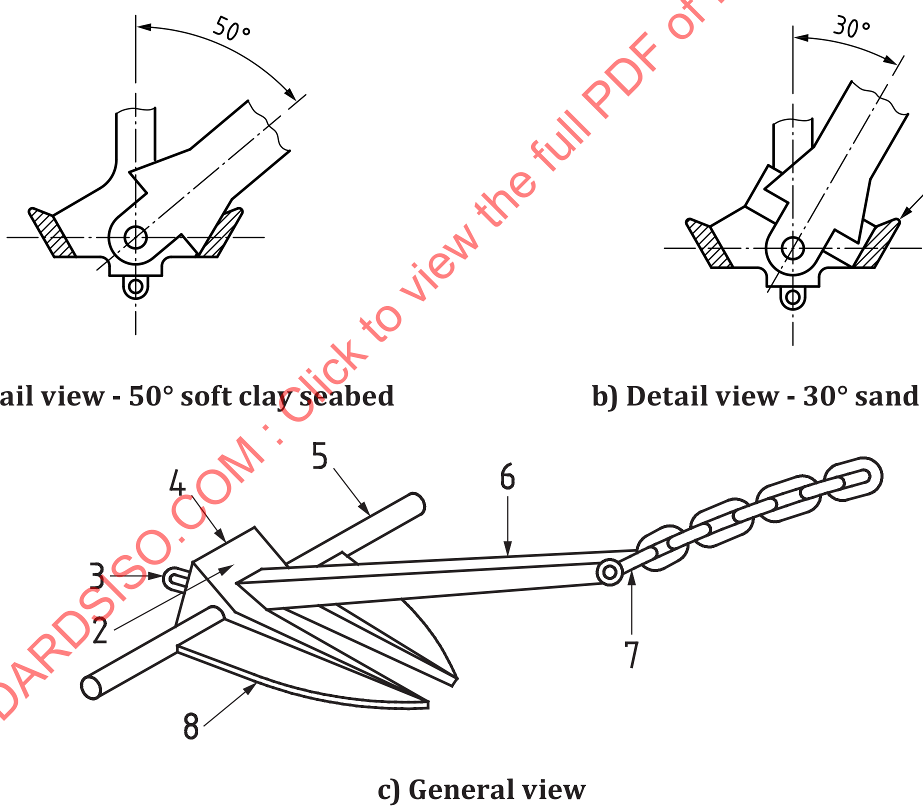
The sampling and *in situ* testing scope and intervals should ensure that each significant stratigraphic layer is properly characterized. The minimum vertical extent of the site investigation should be related to the expected zone of influence of the actions imposed by the base of the foundation and 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.

The content and scope of a deepwater soil investigation should always be tailored to the project-specific conditions. If no previous experience is available for the site, the minimum scope should consist of one boring with alternating sampling and CPTU testing at each anchor cluster. Increasing the number of soil investigation points should be considered if these boreholes show great vertical and/or lateral variability across the mooring pattern. However, if high quality geotechnical data already exist in the general vicinity of the anchor pattern and little variation of soil properties is inferred over the areal extent of the foundation, or if extensive experience with the chosen foundation concept in the area can be drawn upon, the above recommendations can be modified as appropriate.

A.11.3 Anchor types

A.11.3.1 Drag embedment anchors

Traditional drag embedment anchors (Figure A.17) were initially used for mobile (temporary) mooring operations. Drag anchor technology has advanced considerably in recent years. 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 its easy installation and proven performance. The anchor section of a mooring line can be preinstalled and test loaded prior to floating structure installation.



Key

1	adapter block	5	stock
2	tripping palm	6	shank
3	crown padeye	7	shackle
4	crown	8	fluke

Figure A.17 — Traditional drag embedment anchor

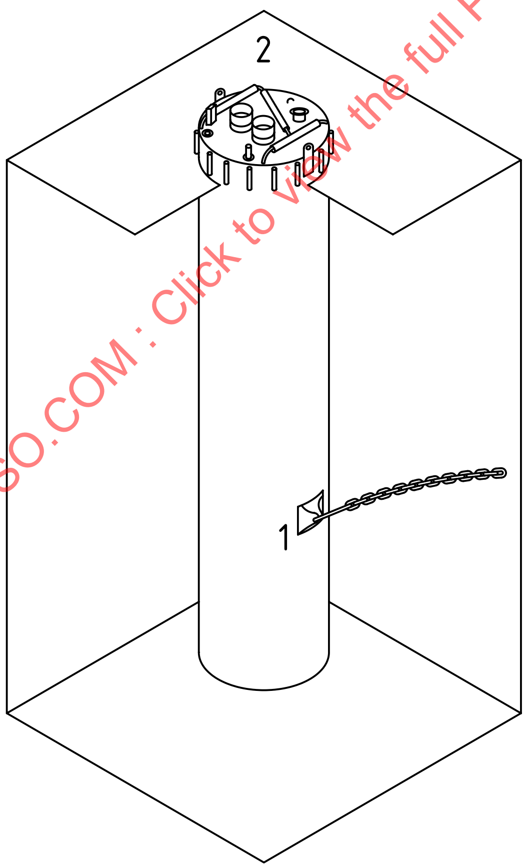
A.11.3.2 Driven anchor piles

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,[A.9-1] jetting,[A.9-25][A.9-26] or drilling and grouting techniques can be considered for other types of anchor piles. However, disturbance of soil during vibro-driving, jetting or drilling operations should be carefully evaluated.

A.11.3.3 Suction anchor piles

Suction anchor piles can be used for large deepwater 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.18). 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 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.18 — 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.

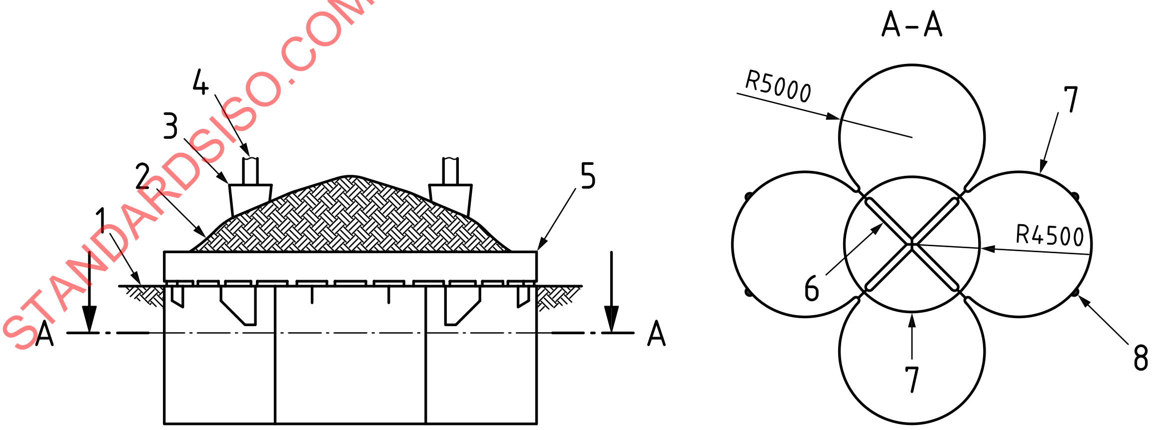
For suction anchor piles embedded in clay and with a closed outlet, the capacity to resist mooring line tensions is governed by the undrained shear strength of the soil around and beneath the anchor. The capacity depends on depth of penetration, anchor diameter, shear strength of the clay, shear strength 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.[A.11-1][A.11-2] Penetration into non-cohesive granular type soils requires special considerations which are not covered here.

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 provide 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 would be an example of suction caisson (see Figure A.19 and Reference [A.11-3]). 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.19 — Suction caisson

A.11.3.4 Plate anchors

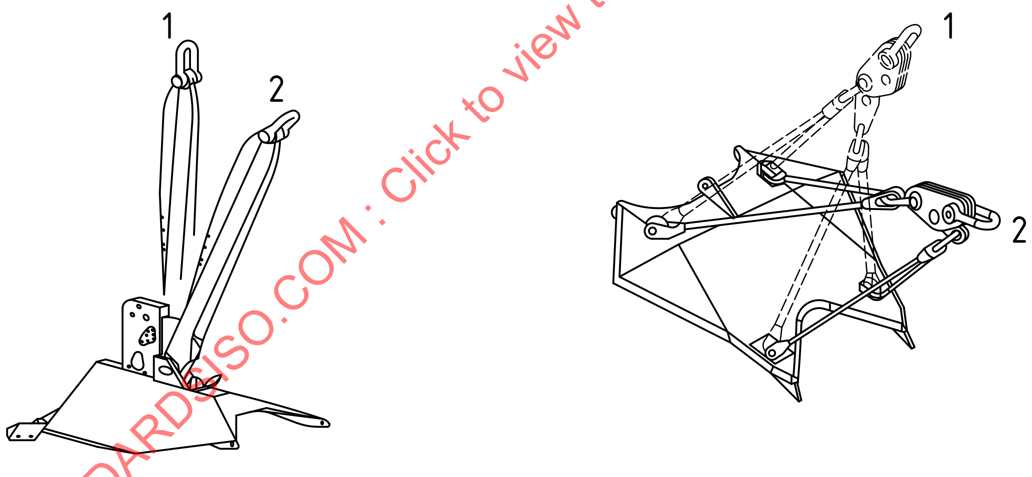
A.11.3.4.1 General

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 that 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[A.11-4] and DENNLA (Drag-Embedded Near Normally Loaded Anchor.[A.11-5] The Stevmanta anchor uses a bridle system to convert from its installation configuration to its plate anchor operational orientation whereas the DENNLA anchor uses an articulated shank (Figure A.20).



a) Dennla Mk4 **b) Stevmanta**

Key

1 normal (or near normal) loading mode

2 installation mode

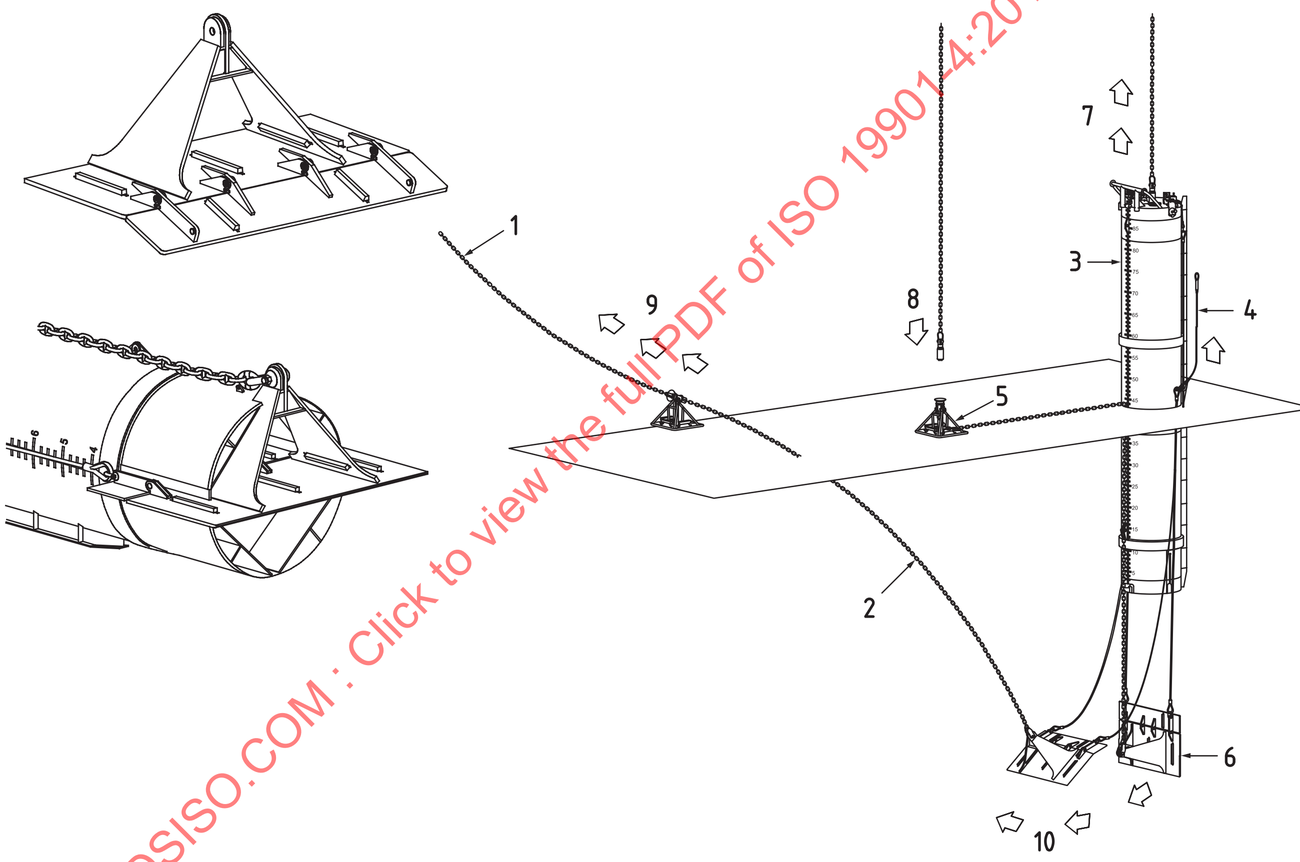
NOTE These anchors are examples of suitable products available commercially. This information is given for the convenience of users of this part of ISO 19901 and does not constitute an endorsement by ISO of these products.

Figure A.20 — 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) 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.21). 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.[A.11-6]



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 part of ISO 19901 and does not constitute an endorsement by ISO of this product.

Figure A.21 — Suction-embedded plate anchor (SEPLA)

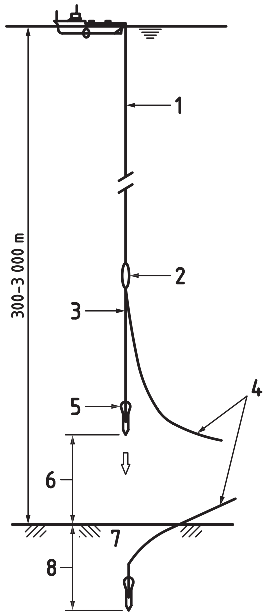
A.11.3.5 Gravity anchors

Gravity anchors are deadweight anchors which commonly consist of concrete or steel blocks, scrap metal or other materials of high density. Penetration results from self-weight and the uplift capacity is dependent on the submerged weight of the anchor. 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 can be used for small mooring systems but typically are not used for large deepwater mooring systems.

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 seabed and penetrate to the target depth below seafloor by kinetic energy obtained during the free-fall (Figure A.22 and References [A.11-7] to [A.11-9]).

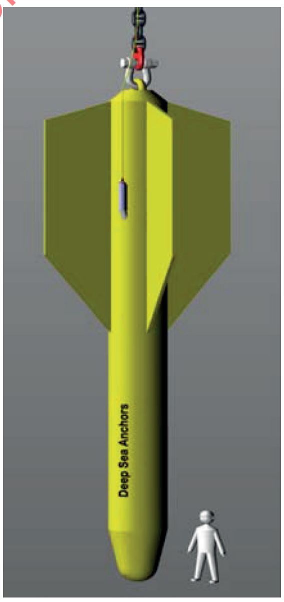
Gravity-embedded anchors obtain significant horizontal and inclined holding capacity by lateral soil resistance against the wings and friction along the soil–steel interface.



a) Installation principle

b) 'Torpedo' pile ([A.11-7])

c) OMNI-Max anchor[A.11-8]



d) Deep penetrating anchor[A.11-9]

Key

1 installation line
2 release unit
3 lead line chain
4 permanent mooring line
5 anchor
6 drop height
7 seabed
8 penetration depth

NOTE These anchors are examples of suitable products available commercially. This information is given for the convenience of users of this part of ISO 19901 and does not constitute an endorsement by ISO of these products.

Figure A.22 — Gravity-embedded free-falling anchors

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 here and in Reference [A.11-10].

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:

— anchor type: fluke area, fluke angle, fluke shape, anchor weight, tripping palms, stabilizer bars, etc. Figure A.23 shows drag anchors commonly used by the offshore industry.

— 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.

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.

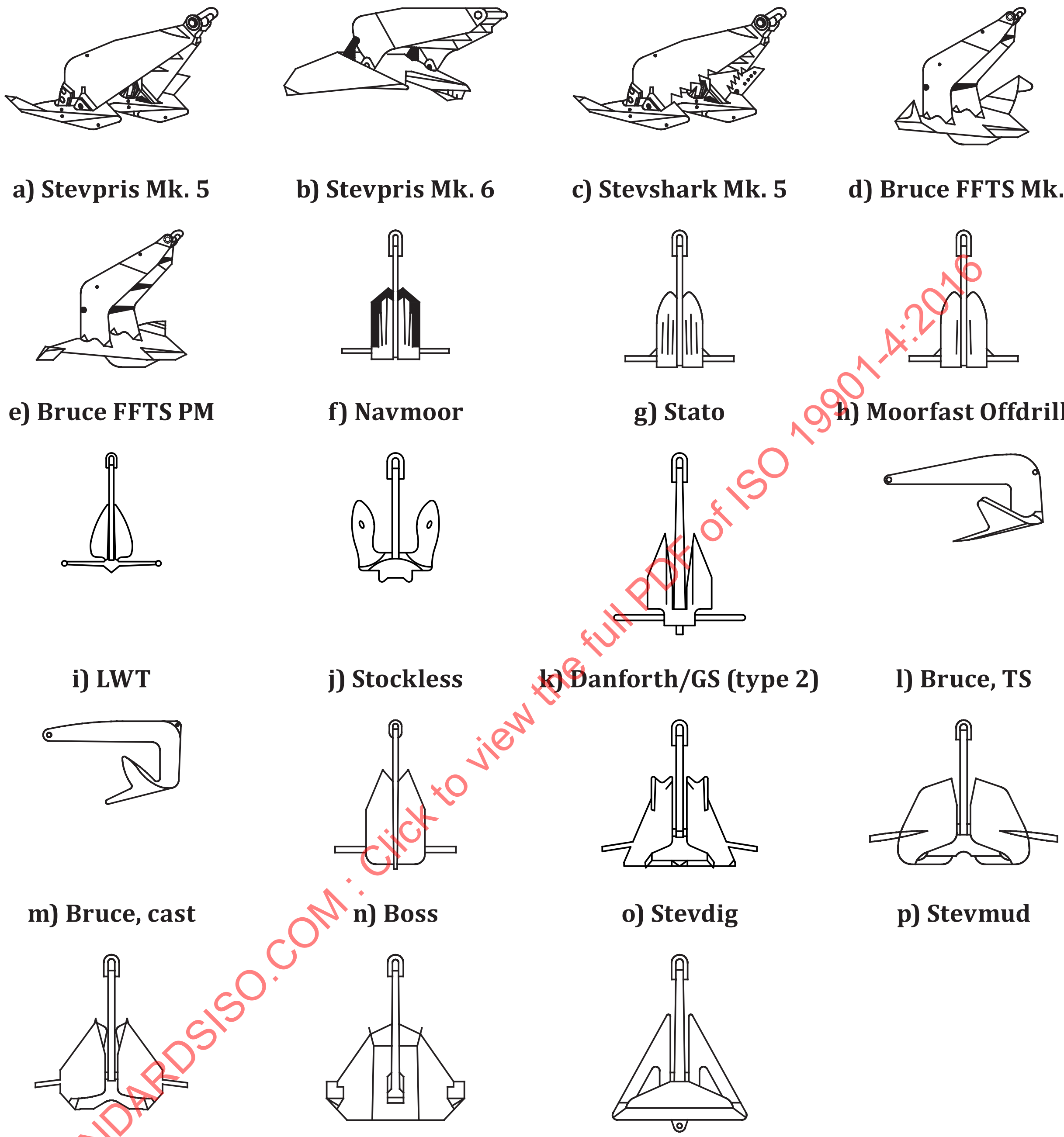
Due to the wide variation of these factors, predicting the holding capacity of a drag anchor is difficult. Exact holding capacity can only be determined after the anchor is deployed and test loaded.

Anchor performance data for the specific anchor type and soil condition should be obtained if possible. In the absence of credible anchor performance data, Figures A.25 and A.26 can be used to estimate the holding power of drag anchors commonly used to moor floating vessels, keeping in mind that the holding capacity curves in Figures A.25 and A.26 do not include a design safety factor.

Figures A.25 and A.26 are reproduced from Reference [5], except that the holding capacity curves for the Moorfast (or Offdrill II) and Stevpris anchors were upgraded, based on model and field test data and field experience. The design curves presented in Figures A.25 and A.26 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.

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 Figures A.25 and A.26.

As Figures A.25 and A.26 only provide anchor holding capacity estimates, more detailed analyses are needed if uncontrolled anchor drag cannot be tolerated in congested subsea locations where it might cause damage to existing subsea installations. If it is impractical to apply an installation tension required to completely avoid future anchor drag, it might be necessary to demonstrate that the extent of anchor drag that can occur will not encroach on the existing subsea installations in the area.



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 part of ISO 19901 and does not constitute an endorsement by ISO of these products.

Figure A.23 — Drag embedment anchors

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.[A.11-12] However, 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 mobilization of the anchor capacity. Of course, this relationship also varies with the anchor type and anchor size.

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.[A.11-13]

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. It should be noted that 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, and 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.

For more information about the prediction of cyclic loading effects, see References [A.11-10], [A.11-14] and [A.11-15]. See also A.8.3.2.3 for more general considerations about the effects of cyclic loading in clay.

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 [A.11-16]. 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, however, be reduced for anchor penetration depths less than 2,5 fluke widths and be set to zero if the fluke penetration depth is very shallow (see further discussion in Reference [A.11-10]).

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.

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 might 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 [A.11-10].

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.

— 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.

— 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.

— A zero uplift angle should be maintained until the recommended minimum anchor penetration depth has been reached.

— 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.5 are applicable for Bruce FFTS Mk. IV and Stevpris Mk. V anchors.

Table A.5 — *R* values for Bruce FFTS Mk. IV and Stevpris Mk. V anchors

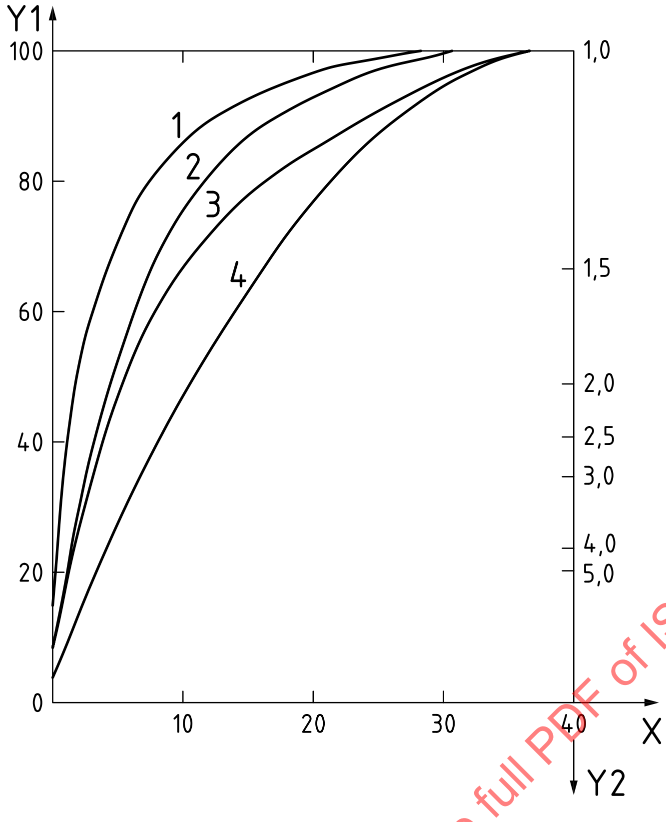
Seafloor angle (°)	0	5	10	15	20
R	1,0	0,98	0,95	0,89	0,81

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 (soil stratigraphy, seafloor shear strength, average shear strength gradient, soil sensitivity, etc.), and the size and type of anchor. For screening-level analysis, drag distance and penetration depth estimates from Reference [A.11-17] are presented in Figure A.24 and Table A.6, 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.



Key

1 stockless anchor (fixed)

2 Hook anchor

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

4 anchor types Boss[a] / Flipper Delta[a] / Stevdig[a] / Stevin[a]

X drag distance/ fluke length

Y1 percent of maximum capacity

Y2 corresponding safety factor

a Assumed based on geometric similarities.

Figure A.24 — Holding capacity versus drag distance in soft clays[A.11-17]

Table A.6 — Estimated maximum fluke tip penetration[A.11-17]

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 mobilized 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

$$H_n = H_s\,(A_n/\,A_s)^n \tag{A.71}$$

where

H_n is the holding capacity of new design;

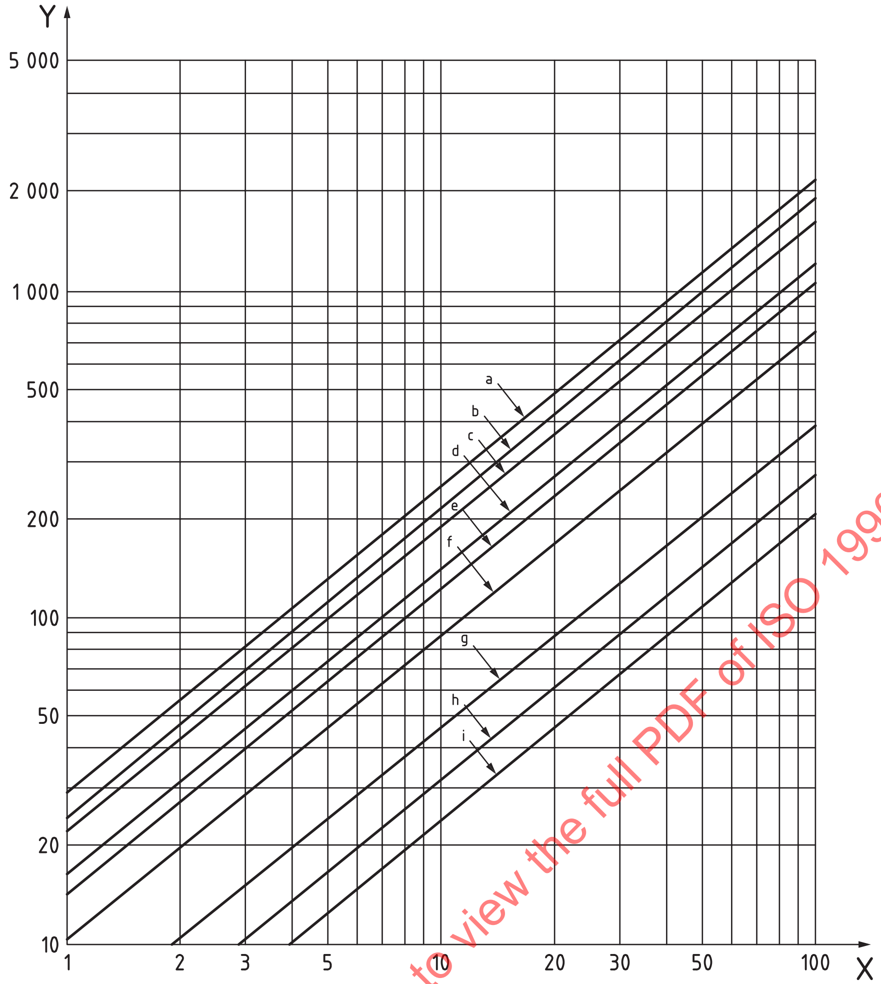
H_s is the holding capacity of reference design (e.g. Bruce FFTS Mk. III or Stevpris Mk. III in Figures A.25 and A.26) of the same weight;

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.



Key

X anchor weight (kips)

Y anchor holding capacity (kips)

Fluke angles set for soft clay seafloor condition as per manufacturer's specification

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

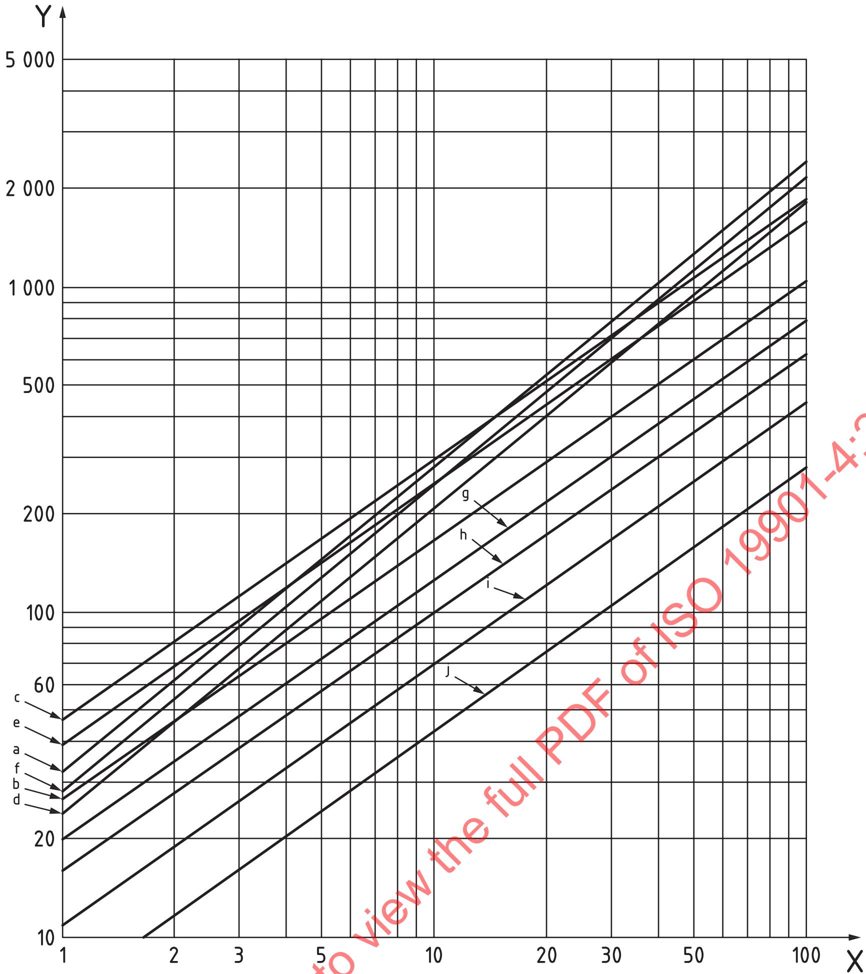
i Stockless, movable fluke

NOTE 1 1 kip = 4,448 kN.

NOTE 2 This Figure was reproduced from Reference [A.11-17], 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 (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 part of ISO 19901 and does not constitute an endorsement by ISO of these products.

Figure A.25 — Anchor system holding capacity in soft clay



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

NOTE 2 This Figure was reproduced from Reference [A.11-17]. 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 (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 part of ISO 19901 and does not constitute an endorsement by ISO of these products.

Figure A.26 — 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, ultimate anchor capacity, etc. However, there are certain requirements for these tools to yield reliable predictions, i.e.:

— 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 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;

— analytical tools should be able to handle layered clay profiles. Some can handle layered clay profiles with sand layers of limited thickness while others 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.[A.11-17] 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.[A.11-10]

In dense sand, anchors that are installed by a mobile offshore drilling unit (MODU) can in some cases still be visible at the seafloor after installation due to the limited capacity of MODU winches. In such cases with shallow penetration anchors, it is not recommended to assume that the anchors continue to penetrate upon overloading. The anchor holding capacity at the achieved penetration depth can be evaluated using the analytical method presented in Reference [A.11-18].

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.

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:

$$P_{cw} = f\, L_{cw}\, W'_{cw} \tag{A.72}$$

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.7 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 [A.11-10]. 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.

Table A.7 — Mooring line friction coefficients

Anchor type	Coefficient of friction f	
	Static	**Sliding**
Chain	1,0	0,7
Wire line	0,6	0,25

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.

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;

— 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:

— drag anchor installation line tension versus time;

— 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;

— direction of anchor embedment;

— drag distance;

— 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.11.5 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 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 the present standard. The recommended design criteria from ISO 19902 and from this part of ISO 19901 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. 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.

As argued in Reference [A.8-75], static p–y curves can be considered for the calculation of lateral soil resistance. Cyclic p–y curves can be more appropriate for fatigue calculations. A modification to the current p–y curves (as described in 8.5 and A.8.5) has been proposed in Reference [A.11-19] to ensure that lateral displacements are not over-predicted. Consideration should be given to degrading the p–y curves for lateral displacements by more than 10 % of the pile diameter. In addition, when lateral displacements associated with cyclic actions at or near the seafloor are relatively large (e.g. exceeding y_c as defined in 8.5 for soft clay), consideration should be given to reducing or neglecting the soil-pile skin friction through this zone.

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 [A.10-2], [A.11-20] and [A.11-21].

A.11.5.1.4 Installation of driven anchor piles

Refer to 9.11.

A.11.5.2 Design of suction anchor piles

A.11.5.2.1 Basic considerations

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).

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 [A.10-16] to [A.10-26] and in References [A.11-11] and [A.11-22].

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 a characteristic 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 might 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. [A.10-28][A.11-23]

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 could 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.

It is of paramount importance to properly estimate the under-pressure (‘suction’) required for the pile to achieve design penetration. 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

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 in clay without protuberances and with a flat tip, the installation resistance at a given tip penetration depth, z, can be calculated by:

$$Q_{tot} = Q_{side} + Q_{tip} \tag{A.73}$$

$$Q_{side} = A_{wall} \left(\alpha_{ins}\, s_{uDSS}\right)_{AVE} \tag{A.74}$$

$$Q_{tip} = (N_c\, s_{utip\ AVE} + \gamma' z)\, A_{tip} \tag{A.75}$$

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, αins

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 but 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 mobilized 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 mobilized 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.[A.10-19][A.10-24] 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, Nc

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 [A.11-24].

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.75) are summarized in Table A.8.

A detailed example of the calculation of N_c is given in Reference [A.11-25]. Values of N_c different from those of Table A.8 are acceptable provided that they can be documented by appropriate modelling and test results.

Table A.8 — 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 (Reference [A.11-20])
Calculation of penetration resistance of protuberances [see A.11.5.2.2.1 item 3)]	Varies	5,0 to 13,5 (Reference [A.10-17])

3) Changes in penetration resistance due to protuberances

Formulae (A.73) to (A.75) 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 considered carefully 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. [A.10-19]

b) Required under-pressure

The required under-pressure, ΔU_{req}, to embed the anchor can be calculated as follows:

$$\Delta U_{\text{req}} = (Q_{\text{tot}} - W') / A_{\text{in}} \tag{A.76}$$

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:

$$\Delta U_{\text{crit}} = N_c\, s_{\text{utip AVE}} + [A_{\text{inside}}\, (\alpha_{\text{ins}}\, s_{\text{uDSS}})_{\text{AVE}}] / A_{\text{in}} \tag{A.77}$$

where

A_{inside} is the inside lateral area of anchor wall.

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 that should 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.[A.11-26]

Soil heave should be accounted for in calculating the required pile stick-up and total length.

e) Presence of sand layers

Sand layers, if present, should be given special attention. The penetration resistance in layered profiles consisting of inter-bedded sands and clays can be significantly higher than through clay, depending on the density, degree of cementation, grain size distribution, and thickness, spacing and depth of the sand layers. Penetration through sand layers is possible, provided the suction anchor design takes this into account.[A.11-27]

The penetration rate through sand layers should be high enough to prevent excessive flow of water through the sand layers ahead of the anchor tip, as this can cause large plug heave.

A.11.5.2.2.2 Removal analysis

The geotechnical analysis should also consider anchor retrieval for the following cases:

— mobile (temporary) moorings where anchor removal is needed for reuse of the anchoring system or to clear the seafloor. The suction pile retrieval procedure and analysis should account for the estimated maximum set-up time;

— permanent moorings where local regulations require removal of the anchors after the structure has reached the end of its service life. The suction pile retrieval procedure and analysis should be based on full soil set-up;

— mobile or permanent moorings where installation tolerances are exceeded, a mooring line is damaged during installation, or for other contingencies.

The extraction pressure required to retrieve the anchor, $(\Delta U_{\mathrm{req}})_{\mathrm{retr}}$, can be calculated by:

$$(\Delta U_{\mathrm{req}})_{\mathrm{retr}} = (Q_{\mathrm{tot}}\ (t = t_{\mathrm{r}}) + W') \,/\, A_{\mathrm{in}} \qquad \text{(A.78)}$$

where

$Q_{\mathrm{tot}}\ (t = t_{\mathrm{r}})$ is the total soil resistance at time of retrieval, t_{r}. Time $t = 0$ is defined as the time at the end of penetration;

W' is the submerged weight of the anchor during retrieval;

A_{in} is the plan view inside area where extraction pressure is applied.

When calculating the total soil resistance during retrieval, $Q_{\mathrm{tot}}\ (t = t_{\mathrm{r}})$, Formula (A.73) can be used with some modifications. It should be noted that the interface shear strength might be higher than its value during installation due to soil set-up. A.11.5.2.2.4 gives guidance on assessing the increase in friction factor with time.

The designer should also be mindful of possible differences between end bearing resistance in tension and compression for protuberances. In addition, the maximum extraction pressure used should not be higher than the pressure causing soil plug failure.

The vessel removing the anchor is often capable of applying a lifting force on the anchor with the recovery line. This assistance can significantly reduce the required extraction pressure and should be included in the removal analysis. Therefore, any loading taken by the lifting line during retrieval can be subtracted from the numerator in Formula (A.78).

The effect of the maximum extraction pressure on the steel structure of the suction pile should also be considered (see A.11.5.2.3).

A.11.5.2.2.3 Holding capacity

Analysis and design tools to determine the capacity of suction anchors can be classified as one of three general methods.[A.10-19] These are, in order of decreasing detail:

— the finite element method (FEM) or other advanced numerical analysis;

— limit equilibrium or plastic limit analysis methods (models involving soil failure mechanisms);

— semi-empirical methods (highly simplified models of soil resistance including beam-column models).

For the analysis and design of suction anchors for anchoring deepwater floaters, the central focus is the ultimate capacity of the suction anchor and not the loading–displacement behaviour.

It is recommended that suction pile designs for permanent moorings use FEM, limit equilibrium techniques or plastic limit analysis. For mobile moorings with mainly horizontal actions, semi-empirical methods such as beam-column analysis using lateral loading or axial shear transfer–displacement curves (i.e. p–y, t–z, Q–z curves described in 8.4 and 8.5), are also considered adequate, if suitably modified. A method to modify p–y curves to account for the larger diameter of suction piles and to ensure lateral displacement is not overestimated can be found in Reference [A.11-19]. The merits and shortcomings of each method are discussed in the following.

a) Finite element method (FEM)

As discussed in Reference [A.10-19], the FEM is the most rigorous general method of analysis available for complex structural systems (including soil continua and soil-structure interaction). The FEM identifies the critical failure mechanism without prior user assumptions, provided an appropriate constitutive model is used. The FEM also has many advantages including the ability to include complex geometries, spatially varying soil properties, and nonlinear constitutive behaviour with failure criterion. Major disadvantages include the required specialist knowledge of advanced numerical analysis and the large time investment to set up a model.

In ductile systems (foundations in soft clays are usually in this category), the ultimate capacity of the system is independent of the sub-failure properties (e.g. Young's modulus, Poisson's ratio, see Reference [A.11-28]). It has been shown that carefully formulated and executed analyses give system capacities that compare favourably with the few exact, analytical solutions available.[A.11-29]

FEM software programs are widely available and have been used to advantage for assessing specific suction pile configurations, matching the few experimental results available and providing calibration of simpler models. Such analyses require special expertise and a significant investment in time and are therefore not yet well suited to parametric studies or conventional design iteration (such as are required for finding the optimum anchor line attachment point, for example).

However, FEM analysis can be warranted for complex loading and/or soil conditions where little experience is available, or to gain insight on specific behavioural aspects of the foundation (i.e. assessment of pore pressure changes and effective stress path at any point within the soil mass).

b) Limit equilibrium or plastic limit analysis methods

As discussed in Reference [A.10-19], these models are more approximate than FEM models but are generally much easier to use than general FEM programs. The methods involve estimating the ultimate capacity of plastic systems using assumed failure mechanisms. These mechanisms are typically based on a combination of experimental observation, more rigorous numerical or analytical studies, and engineering judgment. These methods can also include the ability to

