

Geotechnical and Foundation Design Considerations

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API Foreword

The API Subcommittee on Offshore Structures (SC 2) voted to adopt a modified version of ISO 19901-4:2003 as American National Standard ANSI/API Recommended Practice 2GEO. These modifications from the ISO standard have been incorporated directly into the text and marked with a change bar in the margin.

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Suggested revisions are invited and should be submitted to the Standards Department, API, 200 Massachusetts Avenue, NW, Suite 1100, Washington, DC 20001, standards@api.org.

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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.

International Standards are drafted in accordance with the rules given in the ISO/IEC Directives, Part 2.

The main task of technical committees is to prepare International Standards. Draft International Standards adopted by the technical committees are circulated to the member bodies for voting. Publication as an International Standard requires approval by at least 75 % of the member bodies casting a vote.

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.

ISO 19901-4 was prepared by Technical Committee ISO/TC 67, *Materials, equipment and offshore structures for petroleum, petrochemical and natural gas industries*, Subcommittee SC 7, *Offshore structures*.

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

ISO 19901 is one of a series of standards for offshore structures. The full series consists of the following International Standards.

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

Introduction

The API offshore structures standards 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.

The offshore structures International Standards are intended to provide a wide latitude in the choice of structural configurations, materials and techniques without hindering innovation. Sound engineering judgment is therefore necessary in the use of these International Standards.

The overall concept of structural integrity is described above. 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 behavior 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.

The design practice for the foundations of offshore structures has proved to be an innovative and evolving process over the years since the 1950s. This evolution is expected to continue and is encouraged. Therefore, circumstances can arise when the procedures described herein (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. The scope of the site investigation for one structure is not necessarily adequate for another. Extra caution is necessary when dealing with unfamiliar soils or foundation concepts. This document is intended to provide wide latitude in the choice of site investigation techniques and foundation solutions, without hindering innovation. Sound engineering judgment is therefore necessary in the use of this document.

For an offshore structure and its foundations, 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 may be considered as actions on the foundation's subsystem(s) and this approach is followed in this document.

Some background to and guidance on the use of this document is provided for information in Annex A. Guidance on foundations in carbonate soils is provided for information in Annex B. There is, as yet, insufficient knowledge and understanding of such soils to produce normative requirements.

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

Part 4: Geotechnical and foundation design considerations

1 Scope

This document contains requirements and recommendations 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 characterization,
- soil and rock characterization,
- design and installation of foundations supported by the seabed (shallow foundations),
- identification of hazards,
- design of pile foundations, and
- soil-structure interaction for risers, flowlines, and auxiliary subsea structures.

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

2 Normative references

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

API RP 2A-WSD, 21st Edition, *Recommended Practice for Planning, Designing and Constructing Fixed Offshore Platforms — Working Stress Design*

3 Terms and definitions

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

3.1

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

3.2

effective foundation area

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

3.3

seafloor

interface between the sea and the seabed

3.4**seabed**

materials below the sea in which a structure is founded, whether of soils such as sand, silt or clay, cemented materials or of rock

NOTE 1 The seabed can be considered as the half-space below the seafloor.

NOTE 2 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 rocks. Thus, the term "soil" does not exclude any other material at or below the seafloor.

NOTE 3 As yet there are no universally accepted definitions of the various types of soil and rock, see A.6.4.3.

3.5**settlement**

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

3.6**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

3.7**undrained shear strength**

maximum shear stress or shear stress at a specified shear strain, in an undrained condition

4 Symbols**4.1 General**

Commonly used symbols are listed as follows; 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 stability of shallow foundation

A	actual foundation area
A'	effective foundation area of the foundation depending on the load eccentricity
A_o	actual foundation area per unit length, of an infinitely long footing
A_h	embedded vertical cross-sectional area of foundation
A_p	end area of skirt tip
A_s	side surface area of foundation skirt embedded at a particular penetration depth (including both sides)
$A_{idealized}$	idealized rectangular foundation area, for irregular foundation shapes
b_c, b_q, b_γ	individual 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 the soil over the load range considered

d_c, d_q, d_γ	individual correction factors related to foundation embedment depth
D_b	depth below seabed to base level
e	load eccentricity
e_o	initial void ratio of the soil
e_1	load eccentricity along foundation length
e_2	load eccentricity along foundation width
f	unit shaft friction resistance along foundation skirts, during installation
F	correction factor
g_c, g_q, g_γ	individual correction factors related to foundation seafloor surface inclination
G	elastic shear modulus of the soil
h	soil layer thickness
H	acting horizontal loads, for relevant loading condition
H_d	<i>maximum total horizontal load applied to the base of the foundation at failure under undrained conditions</i>
H_d'	<i>maximum total horizontal load applied to the base of the foundation at failure under drained conditions</i>
H_{ult}	ultimate horizontal capacity in yield surface design method
ΔH	horizontal soil resistance due to active and passive earth pressures on foundation skirts
i_c, i_q, i_γ	individual correction factors related to foundation load inclination
K_c, K_q, K_γ	correction factors, which account for load inclination, footing shape, depth of embedment, inclination of base, and inclination of the seafloor
K_p	<i>coefficient of passive earth pressure</i>
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}	ultimate moment capacity in yield surface design method.
N_c	a dimensionless constant, 5.14

N_q, N_γ	dimensionless functions of ϕ'
p'_o	vertical effective overburden stress at base level (skirt tip level when skirts are used)
q_o	initial effective vertical stress at level of a given soil layer
Δq	added effective vertical stress, at level of a given soil layer
q	unit end bearing pressure on the foundation skirt tip, during penetration
q_c	CPT cone tip resistance, in stress units, for calculation of skirt penetration resistance
Q	acting vertical load, for relevant loading condition
Q_d	maximum total vertical load applied to the base of the footing at failure (<i>excluding</i> weight of soil plug inside skirts) under undrained conditions
Q'_d	maximum total vertical load applied to the base of the footing at failure (<i>excluding</i> weight of soil plug inside skirts) under drained conditions
Q_f	shaft friction resistance along foundation skirts during skirt penetration
Q_o	maximum vertical load, per unit length, of an infinitely long footing
Q_p	end bearing resistance at foundation skirt tip during skirt penetration
Q_r	total soil resistance during skirt penetration
Q_{ult}	ultimate vertical capacity in yield surface design method.
R	radius of the base of a circular foundation
s_u, c_u	undrained shear strength of soil
s_{uo}, c_{uo}	undrained shear strength of soil at foundation base level
$s_{u\text{ ave}}$	average soil strength from seabed to base level
s_{u2}	equivalent shear strength below base level
s_c, s_q, s_γ	individual correction factors related to foundation shape
T	torsional moment
u_v, u_n	vertical and horizontal short-term elastic displacements at foundation base
u_v	vertical long-term settlement of a soil layer
β	ground inclination angle in radians, in calculation of inclination factors
δ	interface friction angle at soil-footing interface (in Clause 7.15)
ϕ'	angle of internal friction of sand for drained triaxial conditions

γ'	effective unit weight of soil
κ	rate of increase of undrained shear strength with depth, for linear shear strength profiles
ν	Poisson's ratio of the soil
ν	base inclination angle in radians, in calculation of inclination factors
θ_r, θ_t	overturning and torsional short term undrained rotations of foundation base

4.3 Symbols for pile foundation design

A_p	gross end area of the pile, $A_p = \frac{\pi \times D^2}{4}$
A_r	pile displacement ratio, $A_r = \frac{A_w}{A_p} = 1 - \left(\frac{D_i}{D}\right)^2$
A_w	cross sectional steel area of pile at pile tip, $A_w = \frac{\pi}{4} \cdot (D^2 - D_i^2)$
A_s	side surface area of pile
C_1, C_2, C_3	Coefficients determined as a function of ϕ'
D	pile outside diameter
D_{50}	mean effective soil particle diameter
D_i	pile inside diameter, $D_i = D - 2 WT$
D_{CPT}	diameter of the CPT tool, $D_{CPT} = 36$ mm for a standard cone with a base area of 10 cm^2
D_r	relative density of a sand; For CPT-based Method 1, $0 \leq D_r \leq 1.0$; for CPT-based Method 4, $D_r > 1.0$ should be accepted and used
E_S	initial modulus of subgrade reaction
$f(z)$	unit shaft friction, in stress units
$f_c(z)$	unit shaft friction in compression, in stress units
$f_p(z)$	unit shaft friction between the sand soil plug and the inner pile wall, for CPT-based Method 4
$f_t(z)$	unit shaft friction in tension, in stress units
h	distance above pile tip $= L - z$
k	rate of increase with depth of initial modulus of subgrade reaction, for lateral soil resistance-displacement p - y curves for sand
K_0	coefficient of lateral earth pressure at rest
L	embedded length of the pile below the original seafloor

L_s	length of the soil plug in the sand layers
N_q	dimensionless bearing capacity factor
p_a	atmospheric pressure, in stress units, (e.g. $p_a = 100$ kPa)
$p'_o(z)$	effective vertical stress at depth z
$p'_{o,tip}$	effective vertical stress at the pile tip
$p'_m(z)$	effective mean stress at depth z
$p'_{oh}(z)$	effective horizontal stress at depth z
P_o	pile outer perimeter = πD
q	unit end bearing at the pile tip, in stress units
$q_c(z)$	CPT cone-tip resistance at depth, z , in stress units
$q_{c,f}(z)$	Reduced CPT cone-tip resistance at depth, z , in stress units, to account for general scour
$q_{c,av,1.5D}$	average value of $q_c(z)$ between $1.5D$ above the pile tip and $1.5D$ below the pile tip
$q_{c,tip}$	cone tip resistance at the pile tip
Q	mobilized end bearing capacity in Q - z curves, in force unit
Q_c	axial pile ultimate capacity in compression, in force units
$Q_{f,c}$	shaft friction capacity in compression, in force units
$Q_{f,t}$	shaft friction capacity in tension, in force units
$Q_{f,i,clay}$	cumulative shaft friction capacity of the clay layers within the soil plug, for CPT method 3
Q_p	end bearing capacity, in force units
Q_t	axial pile ultimate capacity in tension, in force units
s_u	undrained shear strength of the soil at the point in question, in stress units
WT	pile wall thickness
t	mobilized soil-pile adhesion, for axial shear transfer t - z curves
t_{max}	$f(z)$ = maximum soil-pile adhesion or unit shaft friction at depth z , for axial shear transfer t - z curves
t_{res}	residual soil-pile adhesion or unit shaft friction
z	depth below the original seafloor
z	local pile axial deflection, for axial shear transfer t - z curves
z	axial pile tip displacement, for Q - z curves

z'	final depth below seafloor, after general scour
z_{res}	axial pile displacement at which the residual soil-pile adhesion, t_{res} , is reached
α	dimensionless shaft friction factor, for cohesive soils
β	dimensionless shaft friction factor, for cohesionless soils
γ'	submerged, or effective, soil unit weight
ε_c	strain at one-half the maximum deviator stress, for lateral soil resistance-displacement p - y curves for soft clay
δ_{cv}	sand constant volume friction angle at the interface between the sand and the pile wall
ϕ'	angle of internal friction of sand, for drained <i>triaxial</i> conditions
Δz_{GS}	general scour depth
Δz_{LS}	local scour depth
e	base natural logarithms ≈ 2.718
$\ln$	natural logarithm (base e)

5 General requirements

5.1 General

The foundation shall be designed to carry static and dynamic (repetitive as well as transient) loads 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 5.3). Furthermore, the guidance herein does not necessarily apply to so-called "problem" soils such as carbonate material or volcanic sands or highly sensitive clays (see Annex B).

5.2 Testing and instrumentation

Where there is uncertainty regarding the behavior of foundations, testing or instrumentation should be undertaken. Possible methods include the following.

Load testing.

Load testing or large-scale field testing should be performed where there is particular uncertainty in the foundation capacity and where safety and/or economy are of particular importance.

Model tests.

Model tests should be performed where

- 1) the foundation configuration differs significantly from earlier configurations where operational experience exists,
- 2) the soil conditions differ significantly from those where operational experience exists.

- 3) new methods of installation or removal are envisaged, or
- 4) a high degree of uncertainty exists as to how the structure or its foundation will behave.

Temporary instrumentation.

Structures should be fitted with temporary instrumentation where

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

Permanent instrumentation.

Structures should be fitted with permanent instrumentation where

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

5.3 Conductor installation and shallow well drilling

The planning for conductor installation and shallow well drilling shall take into account the potential for disturbance to foundation soils and the consequent risk of a reduction in stability of the structure or of adjacent conductors.

Soil disturbances during drilling operations can result from hydraulic fracture, washout (uncontrolled enlargement of the drilled hole), or 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. Washout generally occurs in granular soils and can, in part, be induced by high drilling fluid circulation rates or drilling without mud. Washout can produce large voids in the soil structure and lead to stress relief in the surrounding soils. These incidents can be accompanied by loss of circulation of drilling fluids, return of these fluids to the seafloor other than through the conductor, or the creation of seafloor craters. Thereby the stability of foundations can be reduced and displacements increased. These detrimental effects can occur whether the drilling takes place after installation of the structure or before, e.g. through a pre-installed template or for an exploration well.

Records of conductor installation and shallow well drilling shall be available to the designer of the structure. The implications for foundation soils of any incidents of inadequate grouting, excessive loss of circulation, return of drilling fluids to the seafloor other than through the conductor, or 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.

6 Geotechnical data acquisition and integrated geoscience studies

6.1 Geotechnical assessment

The determination of geotechnical parameters and the assessment of geological hazards and constraints result from an integrated study of the area using geophysics, geology and geotechnical engineering. 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 are discussed below.

- 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. Seismic three-dimensional data acquired for exploration purposes also provide useful data for developing water-bottom (bathymetry) maps.

These data should only be used for preliminary evaluations because the resolution could be of the order of a few meters depending on the variability of the topography.

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

NOTE These systems can also provide data to develop water-bottom maps.

Side-scan sonar defines seafloor features and seafloor reflectivity.

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

Seismic sources, such as boomers or minisparkers, can define the structure to deeper depths up to approximately 100 m below the seafloor and either 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.

Seabed refraction equipment provides information on the stratification of the top few meters of the seabed.

Shallow sampling of near surface sediments using drop, piston or grab sampling, or vibrocoreing together with cone penetrometer tests along geophysical tracklines can be useful for calibration of results and improved definition of the shallow geology.

Direct observation of the seafloor using a remotely operated vehicle (ROV) or 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

Seismic actions shall be considered in design of structures for areas that are determined to be seismically active. 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 API 2EQ [66]).

Seismic considerations for such areas shall 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 can be subjected to tsunamis shall 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 shall be assessed. If these effects are shown to be detrimental, the magnitude and time scale of expected movement shall be estimated on the basis of a geological study for use in the design of structures.

See Annex A for more guidance.

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 loads.

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 shall focus on 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 can be useful for estimating 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 and waves. 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, as seen in flume models.

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.

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.

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 both gaseous 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 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 shall be taken into account in developing geotechnical parameters for design.

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. Subsidence, where this is a possibility, shall be taken into account in design.

The potential for seabed subsidence due to reservoir compaction shall be considered. The magnitude of surface settlements will depend on the reservoir dimensions, compressibility of reservoir rocks and the anticipated pressure drop. Where the magnitude of subsidence is uncertain, an increase in air gap should be considered.

6.4 Geotechnical investigation

6.4.1 General

Knowledge of the soil conditions existing at the construction site of any structure is necessary to develop a safe and economical design. Site investigations shall be performed to determine the various soil strata and their corresponding physical and engineering properties. Results of previous integrated geoscience studies and experience at the site can enable the design and installation of additional structures with minimal or no additional geotechnical investigation.

The initial step for a geotechnical investigation is a review of available geological, geophysical, and previous soil investigation data. The purpose of this review is to identify potential constraints and aid in planning the subsequent data acquisition phases of the site investigation.

Geophysical surveys should be performed before the geotechnical investigation. Geotechnical surveys and geotechnical investigation data should be combined in an engineering geological model of the region so as to

develop the required design parameters. The on-site studies shall extend throughout the depth and areal extent of soils that will affect or be affected by installation of the foundations.

6.4.2 Soil investigation and testing

The soil investigation program should be defined after review of the geophysical results and the geology at the site. In general, an on-site geotechnical investigation should include

- a) sampling for soil classification and engineering property testing, and
- b) *in situ* soil profiling and strength testing.

Details of the field investigation and subsequent laboratory testing program depend on soil variability, environmental design conditions (e.g. earthquake actions) to be considered in the foundation design, the structure type and geometry, conductor, riser and pipeline requirements, and the presence of geohazards (e.g. slope instability), which can affect the entire construction site.

Depending upon the required investigation depth and sample quality, suitable soil samples can be obtained from geotechnical boreholes, freefall piston corers or vibrocorers. Variations in the vertical soil profile can also be assessed in great detail from continuous piezocone penetration tests (PCPT) and geophysical borehole logging. The PCPT can be performed in a geotechnical borehole or from the seafloor, depending upon the required investigation depth.

In situ strength testing should particularly be considered for investigations where sampling disturbance and/or poor recovery are expected. Sample quality largely depends on the control of the drilling and sampling process, soil type and occurrence of gas in the pore fluid. Sample disturbance and recovery problems are notable in silica sands, carbonate materials and soft soils. *In situ* vane shear test results are used in conjunction with shear strengths measured on retrieved samples to assess soil strength in soft to firm clays. PCPT results combined with the former data can be used to establish a continuous shear strength profile in clays. PCPT data in sands are used to estimate *in situ* relative density, pile and skirt friction and end bearing values.

The objective of laboratory testing is to determine the strength-deformation-consolidation properties of a given soil deposit. Samples should be tested as soon as possible after sampling. A combination of shipboard and land-based laboratory testing is recommended to assess possible transportation, storage, and/or ageing effects.

The required sophistication of the soil sampling and preservation techniques, *in situ* testing, and laboratory testing programs are a function of the structure's design requirements and the need to assess geohazards that can affect the structure. For novel structural concepts, deepwater applications, and structures in areas with potential slope instability, the geotechnical programs should be increased and designed to provide the data necessary for detailed geotechnical analyses. In addition, special geological testing (e.g. X-ray radiography or age dating) can be required to assess geological processes. X-ray radiography can also aid in assessing sample quality prior to extruding samples for laboratory testing.

Geotechnical investigations in seismically active areas should include tests to determine dynamic soil properties and liquefaction potential.

For small structures, the influence depth under the foundation is shallow. The near seafloor soil conditions in soft clay or loose sands and silts are difficult to investigate and are often neglected or misinterpreted. Site investigation tools to be considered for very shallow investigations include, but are not limited to, *in situ* vane tests, cone penetration tests, plate load tests, box core samplers and seabed seismic refraction equipment.

If practical, the soil sampling and testing program should be defined after review of the geophysical results. On-site soil investigation should include one or more soil borings and/or corings to provide samples suitable for engineering property testing and a means to perform *in-situ* testing, if required. The number and depth of borings and/or corings will depend on the soil variability in the vicinity of the site and the platform configuration. Likewise, the degree of sophistication of soil sampling and preservation techniques, required laboratory testing, and the need for *in situ* property testing are a function of the platform design requirements and the adopted design philosophy.

As a minimum requirement, the foundation investigation for pile-supported structures should provide the soil engineering property data needed to determine the following parameters: axial capacity of piles in tension and compression, load-deflection characteristics of axially and laterally loaded piles, pile drivability characteristics, and mudmat bearing capacity.

The required scope of the soil sampling, in-situ testing, and laboratory testing programs is a function of the platform design requirements and the need to characterize active geologic processes that may affect the facility. For novel platform concepts, deepwater applications, platforms in areas of potential slope instability, and gravity-base structures, the geotechnical program should be tailored to provide the data necessary for pertinent soil-structure interaction and pile capacity analyses.

6.4.3 Identification and classification of soils and rocks

Soils and rocks shall be identified and classified in accordance with a recognized published standard (Annex A).

6.4.4 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. Carbonate deposits are variably cemented and range from lightly cemented with sometimes significant void spaces to extremely well cemented. Therefore, in planning a site investigation program, there should be enough flexibility in the program to switch between soil sampling, rotary coring, and *in situ* testing as appropriate. Tests shall be performed to establish the carbonate content. Particularly in sands and silts that contain in excess of 15 % to 20 % carbonate material, foundation behavior can be adversely affected. In these soils, a carefully developed field and laboratory testing program can be warranted (Annex B).

7 Stability of shallow foundations

7.1 General

Recommendations pertaining to the aspects of shallow foundation design described in API 2A-WSD are given in 7.2 to 7.16.

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 are intended for combinations of dead load, variable live load, environmental (i.e. wind, wave and current) load and thermal load (such as arising from pipelines); where these arise from transient or cyclic actions they are considered as quasi-static loads. 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. This includes seismic loading.

These recommendations may also be considered for foundations that are to be used for permanent support. This may be 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 only result in displacements and rotations that could be shown to have minimal environmental, safety and economic consequences. Advice shall be sought from specialist geotechnical engineers where the recommendations in this document are to be considered for permanent foundations.

Alternative design approaches may be more applicable for design of shallow foundations used in more complicated and critical situations, such as for foundations in complex seabed conditions or foundations subject to complex multi-directional cyclic loading conditions (including those where cyclic uplift loads may largely offset the foundation dead weight). This may include use of a partial factor design approach, such as outlined in ISO 19901-4. In particular, advanced analysis techniques such as finite element modeling may be used to confirm that design requirements are met for more complex situations. Additional comments are provided in 7.3.

7.2 Principles

In the following discussion 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.

The following general principles are to be considered in assessing stability of shallow foundations.

- a) Ultimate foundation stability should be analyzed by limit equilibrium methods ensuring equilibrium between applied loads and foundation resistance. Appropriate factors of safety may then be adopted as outlined in 7.3. Due consideration shall be given to potential consequences of excessive displacement and deformation of the foundation soil.
- b) For temporary foundations, design may alternatively be based on displacement (rather than stability) criteria, allowing additional movement under environmental actions. Differential settlement must be considered. The appropriateness of adopting this approach will depend on the type of structure and its installation, and should be the subject of a risk assessment.
- c) If calculation methods other than limit equilibrium are used, an explanation of any differences between calculated foundation capacities and those obtained using limit equilibrium methods should be provided.
- d) For embedded foundations (including surface foundations with skirts) loads acting at base level (typically skirt tip level for skirted foundations) shall be used. Transformation of loads from seafloor to base level shall take account of additional load applied to embedded members, such as that arising from water pressure acting at seafloor level adjacent to the embedded member.
- e) Undrained calculations are typically adopted where no drainage, and hence no dissipation of excess pore pressure, occurs during loading. This may occur as a result of the rate of loading or the impermeable nature of the soil. In contrast, drained calculations are typically adopted where no excess pore pressures arise during loading.

The methods outlined in 7.4 to 7.6 are based on use of the effective area concept. The methods may not be appropriate for use with highly compressible soils or layered soils, and for skirted foundations on soft soils or subject to high overturning moment. Specialist advice shall be sought in these conditions and additional guidance is provided below.

7.3 Acceptance criteria

7.3.1 General

Due consideration of displacement and deformation during the life of the foundation shall be incorporated in the design. Where displacement and deformation are critical, more complex analysis approaches may be warranted.

Foundations designed using the approaches in 7.4, 7.5, 7.6, 7.15, and 7.16 shall have an adequate margin of safety against failure under the design loading conditions. Design loads shall be determined with due consideration of the design life of the foundation.

API 2A-WSD outlines the minimum factors of safety that are recommended for the specific failure modes.

7.3.2 Variations in safety factor

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, increases in the above safety factors may be warranted.

The difference in required minimum safety factor between bearing failure and sliding or torsional failure is intended to account for the relative consequences of each failure mode, and higher factors may be applicable in some situations.

7.3.3 Use in design

7.3.3.1 General

The minimum recommended factors of safety are intended to provide acceptable levels of reliability for shallow foundation design of typical offshore platforms. Safety margins are achieved by reducing the capacity calculated using equations in the relevant section by the relevant safety factor. Note that in some cases (for drained loading) a reduction in vertical load may actually result in a reduction in reliability by making sliding more critical, and care should be taken to ensure an appropriate range in loading (including lowest possible vertical load) is considered in selection of design loads.

It is recommended that the equations are used to derive envelopes that capture the ultimate capacity of the foundation under a range of loading conditions, and that the above safety factors are subsequently applied to these envelopes to derive envelopes of allowable load. Examples are provided in Figure 1 for both undrained and drained conditions, which also illustrate how the safety factors should apply. Note that, in the case of undrained loading, the ultimate capacity envelope is factored by 1.5 against both the vertical load, Q , and the horizontal load, H , axes (not just the H -axis). This is to ensure a minimum 1.5 factor for all load applications and results in a marginal reduction in the allowable load envelope for combinations of high Q and H .

NOTE Since the WSD method is adopted, the maximum storm and operating environmental conditions with no additional load factors should be used to calculate the foundation reaction loading.

Once an envelope of allowable load is derived, the design loads may be transposed onto this envelope to ensure the minimum safety factors are met. As noted in 7.4, 7.5 and 7.6, in the case of skirted foundations the method recommended to calculate the maximum total vertical load at failure assumes there is no difference in the depth of soil inside and outside the skirts. However

- in some cases the soil height above skirt tip may be higher inside the skirt than outside the skirt, such as where significant scour has occurred, or
- in some cases the soil height above skirt tip may be lower inside the skirt than outside the skirt, such as where the foundation has penetrated deeper than the depth of the skirts.

In cases where a difference does exist, this may be accounted for by adjusting the design vertical load. This may be achieved by modifying the design vertical load by the following:

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

where

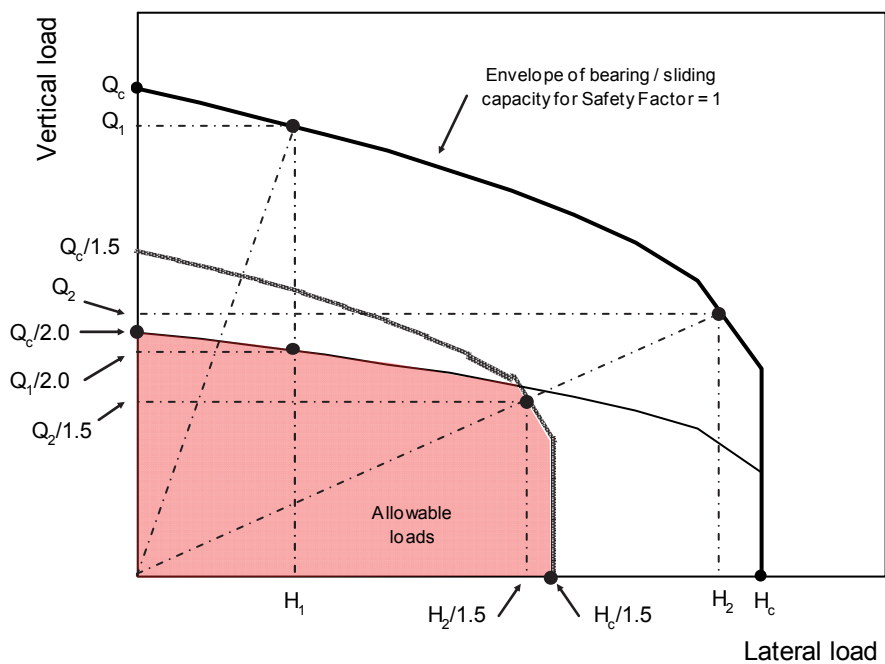
ΔQ is the change in design vertical load to account for differences in vertical effective stress at skirt tip level;

p'_{in} is the *in situ* vertical effective stress at skirt tip level inside the skirts;

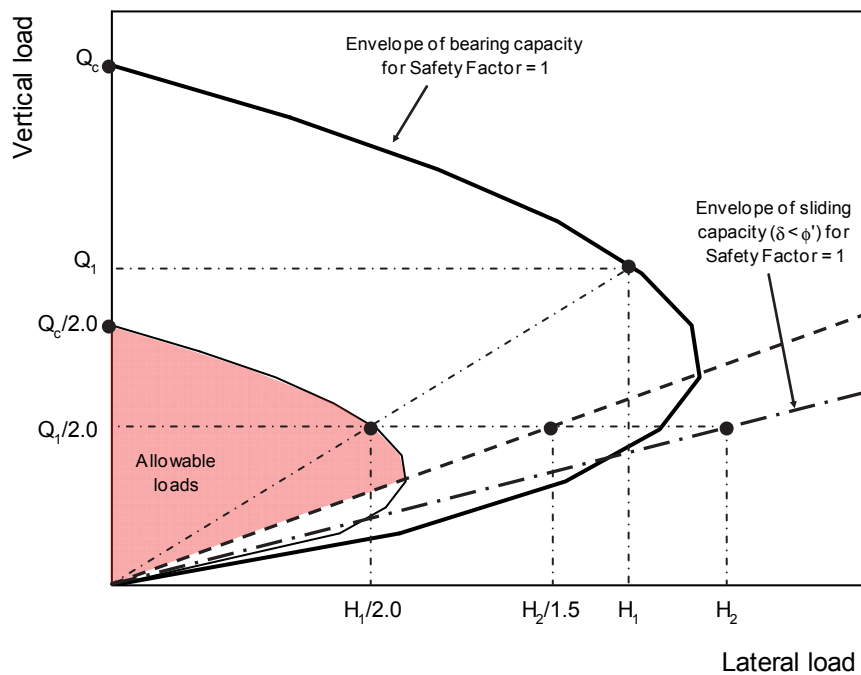
p'_{out} is the *in situ* vertical effective stress at skirt tip level outside the skirts;

A is the actual foundation area.

A similar method may be used for embedded shallow foundations that are designed without skirts. Care should be taken to ensure the design loads are adjusted appropriately in these cases.



Undrained conditions



Drained conditions

Figure 1—Typical envelopes for undrained and drained conditions

These safety factors should be used after the effects of cyclic loading have been taken into account. Discussion of derivation of cyclic soil strength is provided in 7.7.

7.3.3.2 Additional design considerations

The conditions described in 7.3.3.2.1 through 7.3.3.2.6 may require further consideration

7.3.3.2.1 Tensile stresses beneath foundation

Generally, tensile stress (relative to ambient water pressure) beneath skirtless foundations is to be avoided because of the potential disturbance to the underlying seabed due to pumping scour, an erosional mechanism whereby rapid movement of water may lead to undermining of the foundation. A global stability check should be performed to confirm no tensile stress occurs, with the following conditions:

- a minimum safety factor of 1.5 against average soil tension is recommended for individual foundations;
- a minimum factor of safety of 1.0 against localized soil tension anywhere under the foundation is recommended, accounting for flexibility of the foundation.

Tensile stresses within the soil may be acceptable in some cases, such as for skirted foundations that can be shown to resist tension through generation of negative excess pore pressures. Specialist advice shall be sought in relation to design of these foundations. Apart from consideration of the permeability of the soil, drainage paths, duration of action and geometry of the foundation, uplift capacity should be analyzed as a reverse bearing capacity problem with a minimum recommended factor of safety factor equal to 2.0.

7.3.3.2.2 Non-standard soils or soil profiles

The analysis methods outlined above have been developed primarily for use in seabed conditions comprising uniform all sand or all clay profiles. The methods may require more careful consideration when used in other soils, such as silts and carbonate sediments. Specialist *in situ* and laboratory testing may be warranted to determine the characteristics of these materials for use in design. Due care is also required for foundation design in more complex (e.g. layered) soil profiles, such as where 'punch through' of foundations into underlying soils may occur.

7.3.3.2.3 Interaction with other structures

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

7.3.3.2.4 Multiple foundations

For foundations comprising several (connected) foundations, redistribution of loading between individual foundations may be considered, provided the structure can accommodate the resulting distribution, and the soil response is sufficiently ductile.

7.3.3.2.5 Consideration of surrounding seabed conditions

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

7.3.3.2.6 Carbonate soils

Carbonate soils may require particular consideration. Carbonate soils comprise a large percentage of the ocean floor, and for the most part are biogenic (i.e. comprised of the accumulated skeletal remains of plant and animal life). Carbonate soils are susceptible to crushing and are more compressible than (for example) terrigenous silica deposits. This difference from siliceous soils is an important characteristic of carbonate soils which profoundly affects mechanical behavior.

Generally, shallow foundations are suitable for use on carbonate sediments, although any evaluation of such foundations should account for the important difference that exists between such material and 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 pore pressure generation due to cyclic loading. It should also be noted that the undrained cyclic strength of carbonate sands is often lower than that of 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 uncemented sediments should be treated cautiously, with due account taken of the risks and consequences of a punch-through type of failure.

7.3.4 Alternative method of design based on yield surfaces

It is generally accepted that the effective area method is conservative in some instances, especially where large lateral load and overturning moment act in a consistent direction. An alternative method of design involves use of explicit yield functions to derive encompassing yield surfaces directly in vertical, horizontal and overturning moment space. Additional discussion is provided in Annex A.

7.4 Undrained bearing capacity — constant shear strength with depth

When approximately constant undrained shear strength exists to a depth below the base of the foundation equal to at least two-thirds (2/3) of the foundation width, a simplification of constant soil strength may be adopted. However, the impact of lower soil strength below this depth on foundation capacity shall also be considered.

The maximum total vertical load (Q_d) which a footing can support at its base under undrained conditions is

$$Q_d = (s_u N_c K_c) A' \quad (2)$$

where

Q_d is the maximum total vertical load applied to the base of the footing at failure (excluding weight of soil plug inside skirts) under undrained conditions;

s_u is the undrained shear strength of soil (see 7.7);

N_c is a dimensionless constant equal to 5.14;

A' is the effective area of the foundation depending on the load eccentricity;

K_c is the correction factor which accounts for load inclination, footing shape, depth of is the embedment, inclination of base, and inclination of the seafloor.

This method is based on assessment of bearing capacity for surface foundations, or skirted foundations where the weight of the soil plug is balanced by the weight of soil overburden outside the skirts. Where this is not the case, the design load should be appropriately adjusted as outlined in 7.3.3.

A method for determining the correction factor and the effective area is given in Annex A. Two special cases of Equation 2 are frequently encountered. For a vertical central load applied to a foundation at seafloor level where both the foundation base and seafloor are horizontal, Equation 2 is reduced as shown in Equation 3 and Equation 4 for two foundation shapes.

a) Infinitely long strip footing:

$$Q_o = 5.14 s_u A_o$$

where

Q_o is the maximum vertical load per unit length of footing;

A_o is the actual foundation area per unit length.

b) Circular or square footing:

$$Q_d = 6.05 s_u A \quad (4)$$

where

A is the actual foundation area.

7.5 Undrained bearing capacity — linearly increasing shear strength

Seabed sediments can naturally display undrained strength that increases with depth. This clause provides a method to analyze the bearing capacity where the strength can be assumed to increase linearly with depth, and it would be inappropriate to assume a uniform strength.

The maximum total vertical load, Q_d , which a footing can support at its base under undrained conditions is

$$Q_d = F \left(s_{uo} N_c + \frac{\kappa B'}{4} \right) K_c A' \quad (5)$$

where

Q_d is the maximum total vertical load applied to the base of the footing at failure (excluding weight of soil plug inside skirts) under undrained conditions;

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

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

s_{uo} is the undrained shear strength of soil at foundation base level (see 7.7);

N_c is a dimensionless constant equal to 5.14;

B' is the minimum effective lateral foundation dimension;

A' is the effective area of the foundation depending on the load eccentricity;

K_c is the correction factor which accounts for load inclination, footing shape, depth of embedment, inclination of base, and inclination of the seafloor surface.

This method is based on assessment of bearing capacity for surface foundations, or skirted foundations where the weight of the soil plug is balanced by the weight of soil overburden outside the skirts. Where this is not the case, the design load should be appropriately adjusted as outlined in 7.3.3.

A method for determining the correction factors and the effective area is given in Annex A.

7.6 Drained bearing capacity

The maximum total vertical load, Q_d' , which a footing can support at its base under drained conditions is

$$Q_d' = \{p_o' (N_q - 1) K_q + 0.5 \gamma' B' N_\gamma K_\gamma\} A' \quad (6)$$

where

Q_d' is the maximum total vertical load applied to the base of the footing at failure (excluding weight of soil plug inside skirts) under drained conditions;

N_q is $(\exp [\pi \tan \phi']) (\tan^2(45^\circ + \phi'/2))$, a dimensionless function of ϕ' ;

N_γ is an empirical dimensionless function of ϕ' that can be approximated by $1.5(N_q - 1) \tan \phi'$;

ϕ' is the effective friction angle of Mohr envelope (see 7.7);

γ' is the effective unit weight of soil;

p_o' is the vertical effective stress at base level (skirt tip level when skirts are used);

B' is the minimum effective lateral foundation dimension;

A' is the effective area of the foundation depending on the load eccentricity;

K_q, K_γ are the correction factors which account for load inclination, footing shape, depth of embedment, inclination of base, and inclination of the seafloor. The subscripts q and γ refer to the particular term in the equation.

This method is based on assessment of bearing capacity for surface foundations, or skirted foundations where the weight of the soil plug is balanced by the weight of soil overburden outside the skirts. Where this is not the case, the design load should be appropriately adjusted as outlined in 7.3.3.

Equation 6 for drained bearing capacity 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 Annex A.

A complete description of the K factors, as well as curves showing the numerical values of N_q and N_γ as a function of ϕ' are given in Annex A.

Two special cases of Equation 6 are frequently encountered. For a vertical central load applied to a foundation at seafloor level where both the foundation base and seafloor are horizontal, Equation 6 is reduced as follows for two foundation shapes.

a) Infinitely long strip footing:

$$Q_o = 0.5 \gamma' B N_\gamma A_o \quad (7)$$

where

Q_o is the maximum vertical load, per unit length;

B is the minimum lateral foundation dimension.

b) Circular or square footing:

$$Q_d' = 0.3 \gamma' B N_\gamma A$$

7.7 Shear strength used in bearing capacity calculations

The design of shallow foundations is strongly dependent on the quality of the site investigation performed, and the methods used for determining characteristic strength and deformation characteristics, both in situ and in the laboratory. In particular, the uncertainty in determining the characteristic shear strength can be large and either prevent optimization or result in lower than intended safety margins.

Such sources of uncertainty are:

- imprecise definition of the 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.);
- strategies, methods, and procedures used for assessment of the characteristic strength, which vary from one designer or design environment to another;
- amount of testing forming the basis for proposed characteristic shear strengths, statistical variability, and bias.

These uncertainties are relevant for both drained and undrained shear strengths but special care should be taken when the undrained shear strength of sand is applied in the design calculations.

The following general practices are recommended.

- a) For strongly dilatant soils, like dense sand or hard clays, high undrained shear strengths may be used in design only if the potential loss of dilatancy on shear surfaces has been explicitly considered.
- b) For very soft and soft clays, results from unconsolidated undrained triaxial tests should be used with care. Unconfined compression tests should generally not be used without supplementary strength test data. In very soft and soft silty clays, neither unconsolidated undrained triaxial tests nor unconfined compression tests are reliable and should not generally be used. Consolidated undrained triaxial tests with pore pressure measurement, simple shear tests, *in situ* vane, and penetrometer tests (where the correlation between penetration resistance and soil strength is known for a particular soil) are more reliable techniques for determining undrained shear strength of soft clays and silty clays.
- c) Soils display undrained shear strength anisotropy, thus triaxial compression, triaxial extension or simple shear strengths can be different. Care should be taken to adopt an appropriate strength in assessing foundation capacity, and any assumptions made in this regard should be clearly documented.
- d) When undertaking drained bearing capacity calculations for sands using the method outlined in 7.6, the angle of friction (ϕ') should be based on triaxial test conditions. The friction angle used should be derived based on tests performed at appropriate stress levels.
- e) Foundation stability under cyclic loading conditions can be assessed using pseudo-static analysis provided appropriately derived cyclic shear strengths are used.
- f) In many instances cyclic performance of non-cohesive soils may be assessed using cyclic undrained soil strength derived in a manner similar to that used for cohesive soils. This should take into account the potential for dissipation of excess pore pressure to occur over the duration of cyclic loading.
- g) Rate effects should be considered when assessing foundation response to rapid loading events.
- h) Where applicable the effect of soil consolidation on strength may be considered in design. This will typically increase overall foundation capacity.

7.8 Response of shallow foundations to static and pseudo-static loading

The maximum foundation displacement under static or pseudo-static loading may affect the structural integrity of the platform, its serviceability, and its components. Equations for evaluating the static short-term and long-term displacement of shallow foundations are given below. These equations are applicable to idealized conditions, and a discussion of the limitations is given in Annex A.

7.8.1 Short-term displacement (undrained loading)

For foundation materials which 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 deformations of the base under various loads may be estimated as follows:

$$\text{Vertical: } u_v = \left(\frac{1-\nu}{4GR} \right) Q \quad (9)$$

$$\text{Horizontal: } u_h = \left(\frac{7-8\nu}{32(1-\nu)GR} \right) H \quad (10)$$

$$\text{Rocking: } \theta_r = \left(\frac{3(1-\nu)}{8GR^3} \right) M \quad (11)$$

$$\text{Torsion: } \theta_t = \left(\frac{3}{16GR^3} \right) T \quad (12)$$

where

u_v , is the vertical displacement;

u_h is the horizontal displacement;

θ_r is the overturning rotation;

θ_t is the torsional rotation;

Q is the vertical load;

H is the horizontal load;

M is the overturning moment;

T is the torsional moment;

G is the elastic shear modulus of the soil;

ν is Poisson's ratio of the soil;

R is the radius of the base.

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

Equations for rigid, embedded circular foundations are provided in Reference [67] and [68].

Solutions also exist that account for non-uniform soil profiles (e.g. linearly increasing soil strength), general embedded foundations, flexible foundations, and non-uniform base geometries. In addition, consideration should be given to use of numerical analysis methods for more complex situations.

Note that G is not a unique soil parameter and depends on the level of strain applied to each soil element. Care should be taken to select an appropriate (representative) value in design using Equation 9 through Equation 12. Any assumptions made in this regard shall be clearly documented.

7.8.2 Long-term displacement (primary settlement)

An estimate of the vertical settlement of a soil layer under an imposed vertical load can be determined by the following equation:

$$u_v = \left(\frac{hC}{1+e_o} \right) \log_{10} \frac{q_o + \Delta q}{q_o} \quad (13)$$

where

u_v is the vertical settlement;

h is the layer thickness;

e_o is the initial void ratio of the soil;

C is the compression index of the soil over the load range considered;

q_o is the initial effective vertical stress;

Δq is the added effective vertical stress.

Where the vertical stress varies within a thin layer, as in the case of a diminishing stress with depth, estimates may be determined by using the stress at the midpoint of the layer. Thick homogeneous layers should be subdivided for analysis. Where more than one layer is involved, the estimate is simply the sum of the settlement of the layers. Compression characteristics of the soil are determined from appropriate consolidation tests.

7.8.3 Long-term displacement (secondary settlement)

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

7.8.4 Long-term displacement (regional)

Additional settlement may occur in the long-term due to a global lowering of the seafloor due to oil and gas extraction (see 6.3.7). This settlement component is relative to sea (not seafloor) level, and is not significant for temporary foundations.

7.9 Response of shallow foundations to environmental loading

Offshore foundations are subject to environmental loading from current, waves, ice, wind, and earthquakes. In addition, loads may be applied by thermal loading. Both the influence of the foundation on the structural response and the integrity of the foundation itself shall be considered.

In some instances environmental loads may be considered as pseudo-static loads, and assessed using the calculations outlined in 7.8. Examples of where this may be appropriate include current loading.

However, other loading types are cyclic in nature and require more detailed methods of analysis. For example, cyclic wave loading of offshore structures may result in large average and cyclic foundation displacements, as well as volumetric strain after completion of cyclic loading. Specialist advice shall be sought when such analysis is required for foundation design.

7.10 Hydraulic stability

7.10.1 Scour

Positive measures should be taken to avoid erosion and scour of the soil beneath or near the structure base. Examples of such measures are

- scour skirts penetrating through erodible layers into scour resistant materials or deep enough to eliminate the scour hazard, or
- scour protection placed around the edges of the foundation.

Sediment transport studies may be carried out to assess the scour potential of the seabed material around the foundation.

7.10.2 Piping

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

7.11 Installation and removal

Penetration of foundation skirts into the seabed may be facilitated by providing under-pressure (relative to the ambient hydrostatic pressure) inside the skirt compartments under the foundation. In these cases, installation procedures shall be planned to avoid damage 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 they can be accomplished with the means available.

7.12 Shallow foundations equipped with seabed penetrating skirts

For many situations, skirts around the periphery improve the performance of shallow foundations. These skirts penetrate the seabed. If the foundation area is large, interior skirts should also be provided to form skirt compartments under the plate foundation. The presence of the skirts will in most cases

- a) increase foundation resistance to overturning moment, bearing load, sliding load and torsion load,
- b) decrease the vertical and horizontal displacements and rotations, and
- c) improve erosion resistance under the foundation.

In addition to assessing foundation stability, installation studies should be performed to ensure that the skirts can fully penetrate to their intended depth below seafloor. For assessment of bearing capacity, the foundation level and depth of embedment may be considered to be the depth to which the skirts penetrate, provided that it can be shown that governing failure modes do not exist within the skirt compartments.

For assessment of sliding capacity, additional capacity under pure sliding conditions may come from the soil strength at skirt tip level (s_{uo}) being greater than that at seafloor level. Lateral resistance can also be provided above skirt tip level by embedded members, such as through lateral (active and passive) resistance and side shear. This is discussed further in Annex A. Care must be taken to ensure internal mechanisms (within the soil plug for skirted foundations) do not lead to lower overall capacity; while care should also be taken in conditions where installation disturbance or soil conditions may lead to lower resistance above skirt tip level (e.g. in conditions where soil tension cracking on the active side of the skirt may occur).

In addition, skirts may significantly increase the ability of the shallow foundation to sustain short term tensile total stresses (relative to ambient water pressure) beneath the foundation, sustained by negative excess pore pressures within the soil. Cyclic tensile stresses (again, 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 may potentially be carried by skirted foundations on clays with low permeability. Confirmation that tensile stresses can be sustained for a given design should be performed on a case by case basis by specialist geotechnical engineers. The ability of the soil to withstand tensile stresses has been discussed further in 7.3.

Where foundation loads are not provided at the level of the skirt tip(s), but stability analysis is to be performed at this level, care should be taken to appropriately transfer loads to skirt tip. Additional guidance is provided in Annex A.

7.13 Shallow foundations without seabed penetrating skirts

Foundations without skirts, which remain at the seafloor, can require special consideration. In particular, the consequences of loss of contact between the foundation and the seafloor shall be assessed.

7.14 Installation effects

The shallow foundations considered herein 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.

- a) 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 is needed.
- b) 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. Further guidance on this problem is given in Annex A.

7.15 Sliding stability

7.15.1 General

In assessment of sliding stability due consideration shall be given during site investigation and subsequent interpretation to the possible occurrence of discrete layers of low strength soil, which may provide a preferential failure surface.

7.15.2 Surface foundations

When the vertical load capacity has been established by equations in 7.4 to 7.6, the maximum horizontal load should be limited to the capacity determined for the extreme condition of pure sliding as defined by the following equations.

- a) Undrained case:

$$H_d = s_{uo} A \quad (14)$$

where

H_d is the maximum total horizontal load applied to the base of the foundation at failure under undrained conditions;

s_{uo} is the shear strength at base level.

b) Drained case

$$H_d' = Q \tan \phi' \quad (15)$$

where

H_d' is the maximum total horizontal load applied to the base of the foundation at failure under drained conditions;

Q is the actual vertical load acting during the relevant loading condition.

In Equations 14 and 15, it is assumed that full soil resistance may 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.

For drained conditions it may be more appropriate to consider use of a different interface friction angle (δ) between the foundation soil and the structure. In addition, sliding along a thin weak layer of low permeability at shallow depth below the foundation base within the sand mass may need to be assessed.

For undrained cases it may be appropriate to use a soil adhesion coefficient to reduce the undrained strength at foundation level. Alternatively, in cases where an undrained response is anticipated in the bulk of the soil mass it may nevertheless be appropriate to account for a drained interface between the foundation base plate and soil. In addition, the possibility of sliding along a sand seam within a competent clay layer should be considered.

7.16 Torsional stability

Torsional stability requires consideration, and additional analysis shall be performed if torsion loads are significant.

Torsional loads decrease the overall bearing capacity of shallow foundations. Torsional loads may also act to reduce sliding capacity of shallow foundations. Correction factors that account for torsional loads are not available for use with the bearing capacity methods in 7.4 to 7.6, or the assessment of pure sliding in 7.15, and specialist advice shall be sought where this requires consideration.

In assessment of torsional stability 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.

8 Pile foundation design

Recommended design criteria for pile foundations can be found in API 2A-WSD.

8.1 Pile capacity for axial compression

8.1.1 General

Pile capacity for axial compression, as discussed in 8.1.2 through 8.1.5, relates to the axial resistance of a pile when the pile head is subjected to compressive loads 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 through 8.1.5. This simplified model has been developed and applied in many years of offshore practice and represents the current industry standard. However, the model does not provide any information about axial pile displacements which are important for serviceability requirements, especially in non-extreme conditions for loads due to permanent, variable and operating environmental loads that are generally well below the design loads. Axial pile behavior aimed at meeting service requirements is referred to as axial pile performance and is discussed in 8.3. Methods for determining pile performance are described in C.8.6.2 and C.8.6.3.

The simplified model for pile capacity presented in 8.1.2 is based on a (quasi-)static and monotonic application of the axial loads 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).

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 Ultimate axial pile capacity

The ultimate axial capacity of piles in compression, including belled piles, Q_c , should be determined by the equation:

$$Q_c = Q_{f,c} + Q_p = f(z) A_s + q A_p \quad (16)$$

where

$Q_{f,c}$ is the shaft friction capacity in compression, in force units;

Q_p is the end bearing capacity, in force units;

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

A_s is the side surface area of pile;

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

A_p is the gross end area of the pile;

z is the depth below the original seafloor.

For open-ended pipe piles, the total 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 loads in compression on the pile, the weight of the pile shall be considered.

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 C.8.3.

The foundation configurations should be based on those that experience has shown can be installed consistently, practically and economically under similar conditions with the pile size and installation equipment being used. Alternatives for 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 belled piles, the shaft friction values on the pile section should be those given in 8.1 and 8.2. Shaft friction on the upper bell surface and, possibly, on the pile for some distance above the bell should be discounted in computing the shaft 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 Shaft friction and end bearing in cohesive soils

There are a number of methods for calculating the shaft friction and end bearing in cohesive soils. The method described below 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 equations. This matter is discussed below and in C.8.1.3. For pipe piles in cohesive soils, the unit shaft friction, $f(z)$, in stress units, at depth, z , can be calculated using Equation 17.

$$f(z) = \alpha s_u \quad (17)$$

where

α is the dimensionless shaft friction factor, for clays;

s_u is the undrained shear strength of the soil at the point in question, in stress units.

The factor α can be computed by Equation 18:

$$\alpha = 0.5 \psi^{-0.5} \text{ for } \psi \leq 1.0 \quad (18)$$

$$\alpha = 0.5 \psi^{-0.25} \text{ for } \psi > 1.0$$

with the constraint that $\alpha \leq 1.0$

where

$$\psi = \frac{s_u}{p'_o(z)} \text{ at depth, } z; \quad (19)$$

$p'_o(z)$ = effective vertical stress at depth z .

A discussion of appropriate methods for determining the undrained shear strength, s_u , and effective overburden stress, $p'_o(z)$, including the effects of various sampling and testing procedures is included in C.8.1.3. For underconsolidated clays (clays with excess pore pressures undergoing active consolidation), α can usually be taken as 1.0.

Due to the shortage of pile load tests in soils having $s_u/p'_o(z)$ ratios greater than three, Equation 18 should be applied with considerable care for high $s_u/p'_o(z)$ values. Similar judgment should be applied for deep penetrating piles in soils with high undrained shear strength. Low plasticity clays should be treated with particular caution, see C.8.1.3.

For very long piles some reduction in capacity may be warranted, particularly where the shaft friction degrades on continued displacement. This effect is discussed in more detail in C.8.1.3.

Where the pile tip is in cohesive soils, the unit end bearing, q , in stress units, may be computed using Equation 20.

$$q = 9 s_u \quad (20)$$

The shaft 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 shaft friction, the end bearing on the pile wall annulus, and the total internal shaft friction or the end bearing of the plug, whichever is the lesser. For piles considered to be plugged, the bearing pressure may 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 loads.

Shaft 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 during installation and its subsequent dissipation with time. When the design loads 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 behavior is provided in C.8.1.3.

For piles driven in undersized drilled holes, piles jetted in place or piles drilled and grouted in place, the selection of shaft 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 API 2A-WSD, Section 10.4.4.

In layered soils, shaft friction values, $f(z)$, in the cohesive layers shall be as given by Equation 17 through Equation 19. End bearing values for piles tipped in cohesive layers with adjacent weaker layers may be taken as given in Equation 20 provided that

- a) the pile achieves a penetration of two to three diameters or more into the layer in question, and
- c) the tip is approximately three 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 is usually necessary. Where adjacent layers are of comparable strength to the layer of interest, the proximity of the pile tip to the interface is not a concern.

8.1.4 Shaft friction and end bearing in cohesionless soils

This clause provides a simple method for assessing pile capacity in cohesionless soils. Clause C.8.1.4 presents other, recent and more reliable methods. These are based on direct correlations of pile unit friction and end bearing data with cone penetration test (CPT) results. In comparison to the Main Text method described below, these CPT-based methods are considered fundamentally better, have shown statistically closer predictions of pile load test results and, although not required, are in principle the preferred methods. These methods also cover a wider range of cohesionless soils than the Main Text method. However, offshore experience with these CPT methods is limited and hence more experience is needed before they are recommended for routine design, instead of the main text method. 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. Following installation, pile driving (instrumentation) data may be used to give more confidence in predicted capacities.

For pipe piles in cohesionless soils, the unit shaft friction at a given depth, $f(z)$, in stress units, may be calculated by Equation 21.

$$f(z) = \beta p'_o(z) \quad (21)$$

where:

β is the dimensionless shaft friction factor, for sands;

$p'_o(z)$ is the effective vertical stress at depth z .

In the absence of specific data, β values for open-ended pipe piles that are driven unplugged may be taken from Table 1. For full displacement piles (i.e. closed-ended or fully plugged open-ended piles) values of β may 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 Equation 21. In such cases, it may be appropriate to limit f to the values given in Table 1.

Table 1—Design parameters for cohesionless siliceous soil

Relative Density ^a	Soil Description	Shaft Friction Factor ^b β (-)	Limiting Shaft Friction Values kPa (kips/ft ²)	End Bearing Factor N_q (-)	Limiting Unit End Bearing Values MPa (kips/ft ²)
Very loose Loose Loose Medium dense Dense	Sand Sand Sand-silt ^c Silt Silt	Not applicable ^d	Not applicable ^d	Not applicable ^d	Not applicable ^d
Medium dense	Sand-silt ^c	0.29	67 (1.4)	12	3 (60)
Medium dense Dense	Sand Sand-silt ^c	0.37	81 (1.7)	20	5 (100)
Dense Very dense	Sand Sand-silt ^c	0.46	96 (2.0)	40	10 (200)
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 CPT records, strength tests on high quality samples, model tests, or pile driving performance, is available, other values may be justified.					
^a The definitions for the relative density percentage description are as follows: — Very loose, 0 – 15; — Loose, 15 – 35; — Medium dense, 35 – 65; — Dense, 65 – 85; — Very dense, 85 – 100. ^b The shaft friction factor β (equivalent to the “$K \tan \delta$” term used in previous editions of API 2A-WSD) is introduced in this document to avoid confusion with the δ parameter used in the Annex. ^c Sand-silt includes those 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 given in previous editions of API 2A-WSD for these soil/relative density combinations may be unconservative. Hence, it is recommended to use CPT-based methods from the annex for these soils.					

For end bearing of piles in cohesionless soils, the unit end bearing, q , in stress units, may be computed using Equation 22.

$$q = N_q p'_{o, \text{tip}} \quad (22)$$

where

$p'_{o, \text{tip}}$ is the effective vertical stress at the pile tip;

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 Equation 22. In such cases, it is appropriate to limit q to the values given in Table 1. For plugged piles, the bearing pressure may 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 shaft friction on the soil plug equal to the inner unit shaft friction. Note that a pile can be driven in an unplugged condition, but can behave as plugged under static loads.

Load test data for piles in cohesionless soils (see Reference [69] and [70]) indicate that variability in capacity predictions using the simple method described herein can exceed those for piles in clay. These data also indicate that the above method 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 unfamiliar situations, the designer should account for this uncertainty through the selection of conservative design parameters and/or higher safety 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.

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 may require special laboratory or field tests for selection of design parameters. Of particular importance are sands containing calcium carbonate (see A.6.4.4), which are found extensively in many areas of the oceans.

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

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

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

Where these pile tip penetrations are not achieved, some modification in the tabulated values may be necessary. Where adjacent layers are of comparable strength to the layer of interest, the proximity of the pile tip to the layer interface is not a concern.

8.1.5 Shaft friction and end bearing of grouted piles in rock

The unit shaft 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). 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 between the pile and the grout shall be checked in accordance with API 2A-WSD.

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 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 shaft friction and end bearing can be found in various publications (see References [139], [146], and [147]). 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 shaft 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 loads will adversely affect the axial capacity of such piles.

8.2 Pile capacity for axial pullout loads

The pile axial capacity in tension, Q_t , is less than or equal to, but shall not exceed $Q_{f,c}$, the total shaft friction capacity in compression. In computing the tensile design load on the pile, the weight of the pile shall be considered; the weight of the soil plug may also be considered, if this can be justified. For cohesive soils, $f(z)$ shall