INTERNATIONAL STANDARD

ISO 19901-4

First edition
2003-08-01

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

Part 4: Geotechnical and foundation design considerations

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

Partie 4: Bases conceptuelles des fondations

Reference number
ISO 19901-4:2003(E)

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Published in Switzerland

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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 4: Geotechnical and foundation design considerations*

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

The following parts of ISO 19901 are under preparation:

— *Part 1: Metocean design and operating considerations*

— *Part 2: Seismic design procedures and criteria*

— *Part 3: Topsides structure*

— *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 structure*s

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

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

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 judgement 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 behaviour of the seabed, effects from adjacent structures and the results of drilling into the seabed. All of these, and any other relevant information, need to be considered in relation to the overall reliability of the foundation.

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 in the other International Standards ISO 19902 to ISO 19906 (or elsewhere) are insufficient on their own to ensure that a safe and economical foundation design is achieved.

Seabed soils vary. Experience gained at one location is not necessarily applicable at another. 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 part of ISO 19901 is intended to provide wide latitude in the choice of site investigation techniques and foundation solutions, without hindering innovation. Sound engineering judgement is therefore necessary in the use of this part of ISO 19901.

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 part of ISO 19901.

To meet certain needs of industry for linking software to specific elements in this part of ISO 19901, a special numbering system has been permitted for figures, tables and equations.

Some background to and guidance on the use of this part of ISO 19901 is provided for information in Annex A. Guidance on foundations in carbonate soils is provided for information in 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 part of ISO 19901 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), and
- identification of hazards.

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

NOTE 1 Particular requirements for the design of piled foundations, which have a traditional association with fixed steel structures, are given in ISO 19902.

NOTE 2 Particular requirements for the design of shallow gravity foundations, which have a traditional association with fixed concrete structures, are detailed in ISO 19903.

NOTE 3 Particular requirements for the anchor points of mooring systems of floating structures are detailed in ISO 19901-7 [65].

NOTE 4 Particular requirements for the design of spud can foundations, which have a traditional association with jack-up mobile offshore units (MOUs), are detailed in ISO 19905 (all parts).

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.

ISO 19900, *Petroleum and natural gas industries — General 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 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*

3 Terms and definitions

For the purposes of this document, the terms and definitions given in ISO 19900 and the following apply.

3.1
design actions
combination of representative actions and partial safety factors representing a design situation for use in checking the acceptability of a design

3.2
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

3.3
effective foundation area
reduced foundation area having its geometric centre at the point where the resultant action vector intersects the foundation base level

3.4
material factor
partial safety factor applied to the strength of the soil

3.5
sea floor
interface between the sea and the seabed

3.6
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 sea floor.

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 sea floor.

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

3.7
settlement
permanent downward movement of a structure as a result of its own weight and other actions

3.8
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

3.9
undrained shear strength
maximum shear stress at yielding or at a specified maximum strain in an undrained condition

NOTE Yielding is the condition of a material in which a large plastic strain occurs at little or no stress increase.

4 Symbols

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

A total foundation area

A' effective foundation area

A_{h} embedded vertical cross-sectional area of foundation

a soil attraction

B' effective width of foundation

c_{u} undrained shear strength of clay

$c_{\mathrm{u,ave}}$ average undrained shear strength between sea floor and base level for linearly increasing isotropic undrained shear strength with depth

$c_{\mathrm{u,0}}$ undrained shear strength at base level

D_{b} depth to base level

H_{b} factored horizontal total action on base area

K_{c} correction factor, which accounts for inclined actions, foundation shape, and depth of embedment

K_{rd} drained horizontal soil reaction coefficient

K_{ru} undrained horizontal soil reaction coefficient

L effective length of foundation area

p'_0 effective overburden stress at base level (skirt tip level when skirts are used)

$Q_{\mathrm{d,h}}$ design sliding resistance

$Q_{\mathrm{d,v}}$ design bearing capacity in the absence of horizontal actions

$q_{\mathrm{d,v}}$ design unit bearing capacity in the absence of horizontal actions

V_{b} factored vertical total action on base area

γ_{m} material factor

γ' submerged unit weight of soil

κ rate of increase of undrained shear strength with depth

ϕ' effective angle of internal friction

5 General requirements

5.1 General

The foundation shall be designed to carry static and dynamic (repetitive as well as transient) actions without causing excessive deformation of or vibrations in the structure. Special attention shall be given to the effects of repetitive and transient actions on the structural response, as well as on the strength of the supporting soils. The possibility of movement of the seabed shall be considered. Any actions resulting from such movements on foundation members shall be considered in the design. The potential for disturbance to foundation soils by conductor installation or shallow well drilling shall be assessed (see 5.3).

5.2 Testing and instrumentation

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

a) 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.

b) 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.

c) Temporary instrumentation.

   Structures should be fitted with temporary instrumentation where

   1) the installation method presupposes the existence of measured data for control of the operation, or

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

d) Permanent instrumentation.

   Structures should be fitted with permanent instrumentation where

   1) the safety or behaviour of the foundation is dependent on active operation,

      EXAMPLE Where drainage systems are used, data shall be immediately accessible to the user.

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

   3) there is a need for monitoring of the whole foundation with regard to penetration, settlement, tilt, or other behaviour, or

   4) 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 sea floor other than through the conductor, or the creation of sea floor 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 sea floor other than through the conductor, or creation of sea floor craters should be assessed. The cuttings from the well drilling operation, if allowed to accumulate on the sea floor, 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 sea floor morphology. On complex sea floors, 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 metres depending on the variability of the topography.

b) 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.

c) Side-scan sonar defines sea floor features and sea floor reflectivity.

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

d) Seismic sources, such as boomers or minisparkers, can define the structure to deeper depths up to approximately 100 m below the sea floor 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.

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

Shallow sampling of near surface sediments using drop, piston or grab sampling, or vibrocoring 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 sea floor 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 ISO 19901-2 [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 sea floor 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 sea floor should be avoided, if possible. If circumstances dictate siting structures near potentially active faults, 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.

6.3.4 Sea floor instability

Movements of the sea floor 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 sea floor are susceptible to wave-induced movement and can be unstable under very small slope angles. Earthquakes can induce failure of sea floor slopes that are otherwise stable under the existing soil self-weight and wave actions.

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

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

6.3.5 Scour and sediment mobility

Scour is the removal of seabed soils by currents 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 sea floor.

From observations, sea floor 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.

b) Global scour.

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

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

   Such movements can result in sea floor 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 shall 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 programme 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 programme 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 programmes 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 programmes 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 sea floor 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.

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 programme, there should be enough flexibility in the programme 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 behaviour can be adversely affected. In these soils, a carefully developed field and laboratory testing programme can be warranted (Annex B).

7 Stability of shallow foundations

7.1 General

The general equations to be considered when evaluating the stability of shallow foundations are given below and are applicable to idealized conditions. Equations that may be used for detailed design are provided in Annex A together with discussions of the limitations and of alternative approaches. The equations are for static conditions and quasi-static conditions, i.e. taking the maximum environmental action as a static action (monotonic actions only). This is frequently acceptable for small subsea structures, but time-varying actions, if likely to occur, shall be taken into account separately. Where use of these equations is not justified, a more refined analysis or special solutions should be considered. In particular for skirted and/or embedded foundations, see 7.12, the stability during uplift conditions requires special attention with due regard to negative pore pressure (suction), adhesion, permeability of the soil, drainage paths, duration of the action and geometry of the foundation. Unless other failure modes are more critical, the uplift stability may be analysed as a reversed bearing capacity problem applying relevant equations presented below and in Annex A.

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

Suction foundations supporting fixed steel offshore structures may be designed following either the procedure in ISO 19903 or the methodology in this clause. In the latter case higher material factors than those given here can be applicable and appropriate values shall be determined as part of the design.

In this part of ISO 19901, the resistance is computed by applying a material factor to the soil strength. This differs from the practice in ISO 19902, where a resistance factor is applied to the foundation capacity.

7.2 Principles

The foundation stability should be analysed by limit equilibrium methods ensuring equilibrium between design actions and design resistance. Due consideration shall be given to the possibilities of excessive displacement and deformation of the foundation soil.

Calculations using different methods of analysis should include an explanation of any possible differences between calculated material factors.

For an embedded foundation (including surface foundations with skirts), load transfer from sea floor to base level (skirt tip level for skirted foundations) should be calculated as described in Annex A.

In drained conditions, the horizontal and vertical action effects should be assumed as acting on the effective foundation area only. The effective foundation area is defined in Annex A, which also deals with idealization of the foundation area for use with limit equilibrium methods.

In undrained conditions, the action effects may be assumed as being distributed over a greater part or over the total foundation area. In this case, there should be documentation to show that the resulting stress distribution is possible and will not lead to new forms of failure with a lower safety level.

The shape and location of the critical failure zones shall be determined. The shape and location depend on the design actions, the soil stratification, and the applied computation model.

7.3 Acceptance criteria

Numerical methods or the equations presented in 7.4 to 7.6 may be used to develop a soil resistance envelope. Typical examples of such failure envelopes are presented in Figure 1.

The foundation shall satisfy the following criteria.

In soft soils (e.g. normally consolidated clays), foundations will penetrate into the seabed to the depth at which the soil bearing capacity is in equilibrium with the applied bearing pressure. If the foundation is required to provide permanent support, normal practice is either to use skirts or to use ballast and deballast the structure to ensure that the foundation stability requirements are met. For temporary foundations, design may be based on displacement criteria, allowing additional penetration under environmental actions. The allowable uneven (differential) settlement of the structure depends upon the type of structure and its installation and should be the subject of a formal risk assessment. Adequate precautions should be taken to minimize differential settlements between foundations. Good practice should ensure that there is an acceptable limit to the penetration.

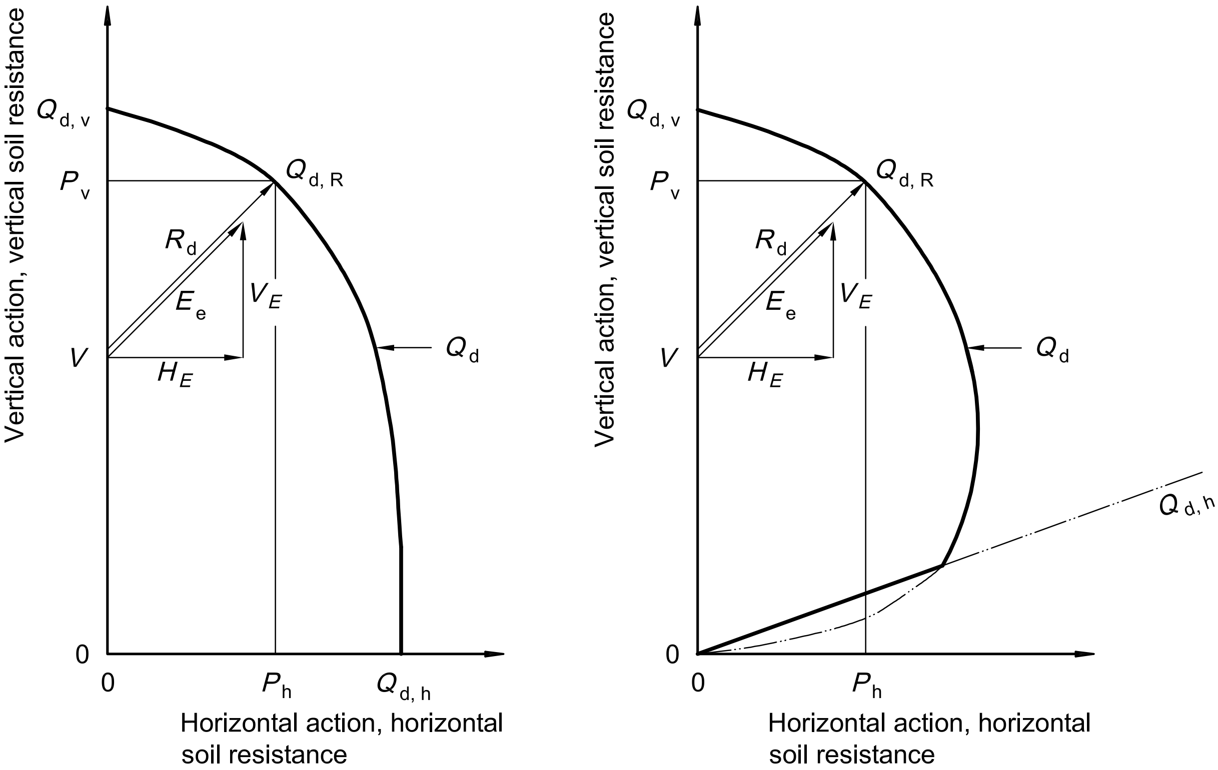
All combinations of horizontal and vertical foundation reactions resulting from different design situations shall satisfy the following criterion:

$$E_e \leqslant R_d \tag{1}$$

where

E_e is the factored environmental action as defined in Figure 1;

R_d is the design soil resistance as defined in Figure 1.



a) Undrained soils **b) Drained soils**

V is the vertical component of the factored permanent and variable actions (see ISO 19900);

E_e is the resultant factored extreme environmental action;

V_E is the vertical component of E_e;

H_E is the horizontal component of E_e;

Q_d is the design soil resistance envelope;

$Q_{d,R}$ is the intersection of the direction of the environmental action vector from V with the design soil resistance envelope;

R_d is the design soil resistance available to resist E_e for a particular V and E_e, and

$$R_d = \sqrt{(P_v - V)^2 + P_h^2}$$

where

P_v is the vertical component of $Q_{d,R}$;

P_h is the horizontal component of $Q_{d,R}$;

$Q_{d,v}$ is the design bearing capacity in the absence of horizontal actions;

$Q_{d,h}$ is the design sliding resistance.

Figure 1 — Soil resistance envelopes

The soil resistance is based on bearing capacity as determined in 7.4, 7.5 or 7.6 using a material factor γ_m = 1,25, except as indicated below or in 7.14.

The above criterion represents the capacity of an individual foundation. For structures comprising several foundations, redistribution of actions between individual foundations may be considered, provided the structure can accommodate the resulting distribution, and the soil response is ductile.

Partial action factors and material factors shall be applied with consistency throughout the design process. In particular, the ratio between the resultant horizontal action and the resultant vertical action on the foundation will influence many of the design equations provided in Annex A. It is important to determine which vertical partial action factor, given in the relevant standard of the ISO 19900 series, produces the more conservative result. The weight of the soil, including that within the skirts, should normally be calculated with factors equal to unity. In some situations, a factor below or above 1,0 can be justified and the actual value shall be determined in each case.

When calculating displacements, all partial action factors and material factors shall be set to unity.

Solutions based on numerical methods should apply the partial action factors as described above. For stability, the computed material factors should equal or exceed those quoted herein.

For situations with negligible environmental actions, the following criterion shall be substituted for Equation (1):

$$V \leqslant Q_{\mathrm{d,v}} \tag{2}$$

where

V is the vertical component of the factored permanent and variable actions;

$Q_{\mathrm{d,v}}$ is the design bearing capacity in the absence of horizontal actions ($= q_{\mathrm{d,v}} A$);

where

$q_{\mathrm{d,v}}$ is the design unit bearing capacity in the absence of horizontal actions calculated using a material factor of γ_m = 1,5; see 7.4 to 7.6.

For situations in which the permanent actions are small, the following criterion may be substituted for Equation (1):

$$H_E \leqslant Q_{\mathrm{d,h}} \tag{3}$$

where

H_E is the horizontal component of the resultant factored extreme environmental action, E_e;

$Q_{\mathrm{d,h}}$ is the design sliding resistance (see A.7.3).

General formulations for computing design unit bearing capacity for some commonly encountered situations are given in 7.4 to 7.6. The special case of horizontal sliding resistance is discussed in Annex A.

The foundations shall have a margin of safety against overturning instability equal to or greater than the minimum required for other modes of failure.

7.4 Undrained bearing capacity — constant shear strength

Equation (4) is a general formula that may be applied for determining design unit bearing capacity for undrained conditions with uniform isotropic undrained shear strength approximately constant with depth under the foundation:

$$q_{\mathrm{d,v}} = N_c \frac{c_u}{\gamma_m} K_c + p_0' \tag{4}$$

where

N_{c} is the bearing capacity factor (see A.7.4.2);

γ_{m} is the material factor from 7.3;

K_{c} is a correction factor, which accounts for inclined actions, foundation shape, and depth of embedment (see A.7.4).

NOTE For vertical actions on a surface strip, $K_{\mathrm{c}} = 1$.

Equation (4) applies to situations with approximately uniform undrained shear strength to a depth equal to at least 2/3 of the foundation width. For situations with low undrained shear strength immediately under the footing and increasing with depth, very shallow bearing capacity failure mechanisms can occur. Thus, the "representative value" of undrained shear strength c_{u} in Equation (4) can be seriously overestimated. The results of conventional slip circle analyses can also seriously overestimate the bearing capacity for such situations; 7.5 deals with this situation.

7.5 Undrained bearing capacity — linearly increasing shear strength

Equation (5) is a general formula that may be applied for determining design unit bearing capacity for undrained conditions with uniform isotropic undrained shear strength increasing approximately linearly with depth under the foundation:

$$q_{\mathrm{d,v}} = F\left(N_{\mathrm{c}} c_{\mathrm{u,0}} + \frac{\kappa B'}{4}\right)\frac{K_{\mathrm{c}}}{\gamma_{\mathrm{m}}} + p'_0 \tag{5}$$

where

F is a correction factor given as function of $\kappa B'/c_{\mathrm{u,0}}$ (see A.7.5);

N_{c} is the bearing capacity factor (see A.7.4.2);

K_{c} is a correction factor, which accounts for inclined actions, foundation shape, and depth of embedment (see A.7.5);

γ_{m} is the material factor from 7.3.

7.6 Drained bearing capacity

Equation (6) is a general formula that may be applied for determining design unit bearing capacity for drained conditions:

$$q_{\mathrm{d,v}} = 0{,}5\gamma' B' N_\gamma K_\gamma + (p'_0 + a) N_{\mathrm{q}} K_{\mathrm{q}} - a \tag{6}$$

where

N_γ, N_{q} are bearing capacity factors that include material factors (see A.7.6);

K_γ, K_{q} are correction factors that account for inclined actions, foundation shape, and depth of embedment, which include material factors where appropriate (see Note);

a is the soil attraction and $a = c' \cot\phi'$;

where

c' is the cohesion intercept.

NOTE For actions on a surface strip, both correction factors equal unity (see A.7.6).

7.7 Shear strength used in bearing capacity calculations

The safety of shallow foundations is strongly dependent on the quality of the site investigation 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 lead either to uneconomical conservatism or to lower than intended safety margins.

Such sources of uncertainty are

a) imprecise definition of the characteristic strength,

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

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

d) 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.

For strongly dilatant soils like dense sand or hard clays, high undrained shear strengths may be accepted, provided the deformations are acceptable and the possible loss of dilatancy for actual conditions has been checked.

In soft clays, results from unconsolidated undrained triaxial tests should be used with care. Unconfined compression tests should generally not be used. In soft silty clays, neither unconsolidated undrained triaxial tests nor unconfined compression tests are reliable. Consideration should be given to consolidated undrained tests.

In drained bearing capacity calculations on sands using Equation (6), the effective plane strain angle of friction determined as a tangent parameter should be used. This value should be determined at the appropriate stress level (see A.7.6 and A.7.7).

7.8 Settlements and displacements

Displacements over the life of the installation shall be determined and taken into account in determining the required air gap and other serviceability limits.

The foundation settlement calculations shall include

a) immediate settlements,

b) consolidation settlements of the soil (long-term soil deformation),

c) soil settlements induced by cyclic actions,

d) differential settlements induced by moments, torque and eccentricity, and

e) creep.

7.9 Dynamic behaviour

Dynamic actions are imposed on a structure-foundation system by current, waves, ice, wind, and earthquakes. Both the influence of the foundation on the structural response and the integrity of the foundation itself shall be considered.

7.10 Hydraulic stability

7.10.1 Scour

Positive measures to prevent erosion and undercutting of the soil beneath or near the structure base due to scour should be considered. Possible methods include

a) scour skirts penetrating through erodible layers into scour-resistant soils or to such depths as to eliminate the scour hazard, and

b) scour-resistant materials placed around the edges of the foundation.

Sediment transport studies can be of value in planning and design. Alternatively, the foundation may be designed to tolerate erosion of part or all soils that are not scour-resistant.

7.10.2 Piping

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

7.11 Installation and removal

Installation shall be planned to ensure that the foundation can be properly seated at the intended site without excessive disturbance to the supporting soil. If removal is anticipated, an analysis should be made of the actions generated during removal to ensure that removal can be accomplished with the means available.

7.12 Shallow foundations equipped with skirts

For many situations, it is advantageous to equip the foundation plate with vertical skirts around its periphery that penetrate into the seabed. If the foundation area is large, interior skirts should also be provided to form skirt compartments under the plate foundation. The presence of the skirts will in most cases

a) increase the capacity of the foundation to resist vertical, horizontal, and overturning actions,

b) decrease the vertical and horizontal displacements and rotations, and

c) improve erosion resistance under the foundation.

Skirts significantly increase the ability of shallow foundations to sustain short-term tension. The uplift causes pore-water underpressure below the foundation plate and consequently the tension capacity reduces as the underpressure dissipates. Cyclic uplift from ocean waves with a few seconds duration can safely be resisted, even on relatively pervious sand deposits. Uplift of longer duration can be carried by skirted foundations on clays with lower permeability.

During foundation installation, the skirt penetration into the seabed may be facilitated by providing underpressure (relative to the ambient hydrostatic pressure) inside the skirt compartments under the foundation. This can be necessary for skirted foundations on dense sand and stiff clays. Procedures shall be designed to avoid piping.

7.13 Shallow foundations without skirts

Foundations without skirts, which remain at the sea floor, can require special consideration.

The consequences of loss of contact between the foundation and the sea floor shall be assessed.

7.14 Installation effects

The shallow foundations considered herein are often placed on the sea floor 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 sea floor 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 sea floor ($< 0,2$ 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.

Annex A
(informative)

Additional information and guidance

NOTE The clauses in this annex provide additional information and guidance on clauses in the body of this part of ISO 19901. The title of each subclause identifies the subclause in the body of this part of ISO 19901.

A.1 Scope

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

— the Proceedings of the Offshore Technology Conference (OTC), Houston, Texas, which is held annually;

— the Proceedings of the International Conference on the Behaviour of Offshore Structures (BOSS), which is held at intervals of approximately 3 years;

— the Proceedings of various Conferences of the Society of Underwater Technology, London, UK, held at less regular intervals.

A review of the practice of the subjects covered by this annex may be found in Reference [1] of the Bibliography. General guidance on the application of soil mechanics theory to foundation design may be found in undergraduate and post-graduate textbooks, such as References [2] and [3], respectively.

A.2 Normative references

No guidance is offered.

A.3 Terms and definitions

No guidance is offered.

A.4 Symbols

No guidance is offered.

A.5 General requirements

No guidance is offered.

A.6 Geotechnical data acquisition and integrated geoscience studies

A.6.1 Geotechnical assessment

Further information on using the results of integrated geoscience studies to perform a geotechnical assessment may be found in References [4] and [5].

A.6.2 Shallow geophysical investigation

No guidance is offered.

A.6.3 Geological modelling and identification of hazards

A.6.3.1 General

No guidance is offered.

A.6.3.2 Earthquakes

No guidance is offered.

A.6.3.3 Fault planes

No guidance is offered.

A.6.3.4 Sea floor instability

No guidance is offered.

A.6.3.5 Scour and sediment mobility

No guidance is offered.

A.6.3.6 Shallow gas

No guidance is offered.

A.6.3.7 Seabed subsidence

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

A.6.4 Geotechnical investigation

A.6.4.1 General

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

A.6.4.2 Soil investigation and testing

Laboratory and *in situ* tests should be performed in accordance with procedures described in applicable national or International Standards, if available, which should be referenced in all relevant documentation.

The vertical soil profile can be assessed using continuous profile piezocone penetration test (PCPT) data and empirical correlation charts that relate PCPT results to soil classification[7]. Site-specific correlation charts, similar to the ones presented in Reference [8], can also be developed for this purpose. PCPT data can also be used to estimate trends in soil strength (for clays) and relative density (for sands) at the site. Borehole geophysical logging techniques can prove useful in identifying changes in strata and soil properties[9]. Consideration can be given to performing vertical seismic profiling in geotechnical boreholes or in seabed PCPTs for calibration of seismic survey data[10] [11] [12].

A general overview of offshore soil investigation techniques is given in References [13] and [14]. Recent advances, particularly related to deepwater investigations are described in References [15], [16] and [17]. Numerous *in situ* testing techniques other than PCPT and vane shear testing are available for determining *in situ* strength and other soil parameters. A comprehensive overview of such tests is given in Reference [18].

Significant scatter in the results of conventional laboratory shear tests can occur when performed on samples of silty clays and clayey silts. More consistent strength profiles can be achieved using the "Stress history and normalized soil engineering properties" (SHANSEP) soil strength testing described in References [19] and [20]. This methodology can be used to assess shear strength profiles in cohesive soils, as this technique addresses the critical variables that influence soil behaviour, including

a) anisotropy,

b) sample disturbance,

c) stress history changes,

d) coefficient of earth pressure at rest, and

e) rate effects.

A.6.4.3 Identification and classification of soils and rocks

Soils and rocks should be completely identified and classified with reference to a consistent classification system[21] [22] [23]. ISO 14688-1, ISO 14688-2 and ISO 14689-1 address identification and classification of soils[24] [25] and rocks[26].

Three groups of parameters are required to completely identify a soil as follows:

a) parameters necessary to define the nature of the soil, e.g. grain size, Atterberg limits, carbonate content, organic content, specific density of grains;

b) parameters related to the *in situ* state of the soil, e.g. water content, submerged unit weight, consistency of clays, relative density of sands;

c) mechanical parameters, e.g. parameters giving the strength and deformation properties of the material.

These parameters, with the exception of relative density of sands, can be obtained from laboratory tests. The soil should then be categorized according to predefined soil groups, assigned a group symbol, and thereby classified.

No universally recognized classification system is presently available for carbonate materials. A classification chart for the carbonate soils and rocks encountered in the Middle East area has been tentatively developed[27]. It is based on grain size, carbonate content, and unconfined compressive strength of materials. It is recognized today that parameters such as grain crushability or skeleton compressibility play an important role in assessing the engineering properties of carbonate materials. However, in the absence of a more definite classification scheme, the proposed chart can provide useful guidance.

The assessment of the *in situ* relative density of sands can be of particular importance for foundation design. Common practice in the offshore industry is to use empirical correlations between the relative density of siliceous sands and cone resistance as obtained from cone penetration tests. Because of the significant effects of the *in situ* stress conditions, correlations for a representative range of stress conditions should be used[28] [29] [30]. These correlations involve the use of an element of subjective judgement. In situations where this will have a significant impact on the reliability of the foundations, an alternative design analysis should be made using design procedures based directly on the cone resistance. If such procedures are not available for the particular situation being addressed, the alternative of a direct measurement of density *in situ* may be considered.

In assessing the quality and properties of rocks and rock masses, a distinction should be made between the behaviour of rock material as measured on core samples and the behaviour of a much larger rock mass that is affected by structural discontinuities, such as bedding plates, joints, shear zones, or solution cavities.

Useful guidance for classifying rocks may be found in the recommendations of the International Society for Rock Mechanics[31]. The following characteristics should be identified:

— origin (e.g. sedimentary, volcanic),

— strength of rock matrix,

— degree of alteration of the rock mass, and

— state of fracturation of the rock mass.

The rock quality may be quantified using the rock quality designation (RQD), which is an indication of the quality of the rock mass for engineering purposes.

A.6.4.4 Carbonate soils

Information on carbonate soils is given in Annex B.

A.7 Stability of shallow foundations

A.7.1 General

The equations provided herein are limited and are not necessarily appropriate for design in a number of situations, particularly if the soil exhibits contractancy. A common situation for which they cannot be applied arises when there is a strong layer overlying a weak layer within the zone of influence of the foundation. Irregularly shaped foundations can be difficult to analyse using the equations.

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

The bearing capacity factors used herein are considered the most commonly used, but alternative factors, particularly for circular foundations, are available and may be applied at the discretion of the designer.

A.7.2 Principles

A.7.2.1 Load transfer

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

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

b) foundation weight, and

c) soil weight within skirts (if applicable).

Partial action factors should be applied to the foundation weight and soil weight. The factor for soil weight is generally equal to unity, however higher or lower factors should be considered in each case.

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

— horizontal resistance on the skirts which reduces the external horizontal and moment actions, and

— frictional resistance on the skirts, which reduces the external vertical and moment actions.

Material factors should be applied when computing these soil resistances. The frictional resistance can be determined from ISO 19902.

For the undrained case, the horizontal resistance, ΔH, between sea floor and base level can be approximately calculated as follows:

$$\Delta H = K_{\text{ru}} \left(c_{\text{u,ave}} / \gamma_{\text{m}} \right) A_{\text{h}} \tag{A.1}$$

The undrained horizontal soil reaction coefficient K_{ru} depends on several factors, such as roughness, foundation shape, side shear, depth of embedment, and possible side gap between foundation and soil due to installation or from scour. Provided that the installation procedure and/or other foundation aspects do not require a more accurate assessment of the undrained horizontal soil reaction factor, K_{ru} = 4 is recommended.

For the drained case, the horizontal resistance between sea floor and base level can be approximately calculated as follows:

$$\Delta H = K_{\text{rd}} (0{,}5\gamma' D_{\text{b}} + a) A_{\text{h}} \tag{A.2}$$

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

$$K_{\text{rd}} = K_{\text{p}} - \left(1 / K_{\text{p}} \right) \tag{A.3}$$

where

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

In the following subclauses the factored vertical and horizontal total actions on the base area are denoted V_{b} and H_{b} respectively.

A.7.2.2 Idealization of foundation area and the effective area concept

If the foundation stability problem is solved by using three-dimensional finite element analysis, the foundation area can be modelled with no or with only minor simplifications of the geometry. Limiting equilibrium methods are generally based on a two-dimensional model (vertical slice) where the three-dimensional effects are included by defining the resistance of the vertical side areas. This requires a rectangular idealization of the foundation area. This idealized area can be defined by a rectangle of width B and length L having the same area, A, and the same areal moments of inertia, I_x and I_y, as for the real area:

$$A_{\text{idealized}} = BL = A_{\text{real}} \tag{A.5}$$

$$I_{x,\text{idealized}} = I_{x,\text{real}} \quad \text{for action effects in the } y\text{-direction} \tag{A.6}$$

$$I_{y,\text{idealized}} = I_{y,\text{real}} \quad \text{for action effects in the } x\text{-direction} \tag{A.7}$$

The width B and length L of the idealized foundation area are determined by solving Equations (A.5) to (A.7).

The effective foundation width B' and the consequent effective foundation area A' are used to determine the bearing capacity correction factors and the sliding resistance. The effective width B' normal to the plane of the design actions is determined from Equation (A.8):

$$B' = B - 2e \tag{A.8}$$

where

e is the eccentricity of the vertical component of the design actions, i.e. the distance between the centre of action of the total design actions and the vertical components of the design actions at foundation base level. When skirts are used, base level is taken as skirt tip level.

The corresponding effective foundation area A' is:

$$A' = B'L \tag{A.9}$$

A.7.3 Acceptance criteria

A.7.3.1 Sliding resistance for constant undrained shear strength

In the absence of vertical actions, the design sliding resistance at the base of the foundation is determined through:

$$Q_{\mathrm{d,h}} = (c_{\mathrm{u}}/\gamma_{\mathrm{m}})A \tag{A.10}$$

A.7.3.2 Sliding resistance for linearly increasing undrained shear strength

In the absence of vertical actions, the design sliding resistance at the base of the foundation is determined through:

$$Q_{\mathrm{d,h}} = (c_{\mathrm{u,0}}/\gamma_{\mathrm{m}})A \tag{A.11}$$

A.7.3.3 Sliding resistance for drained conditions

In the absence of significant vertical actions, the design sliding resistance at the base of the foundation is determined through:

$$Q_{\mathrm{d,h}} = \left(V_{\mathrm{b}} + aA'\right)\left(\frac{\tan\phi'}{\gamma_{\mathrm{m}}}\right) \tag{A.12}$$

A.7.4 Undrained bearing capacity — constant shear strength

A.7.4.1 Bearing capacity formulae

With regard to the bearing capacity equations presented in 7.4 the following bearing capacity factors and correction factors primarily come from References [32] to [36]. In general, these equations and factors should be used with care and their applicability should be checked in each case.

A.7.4.2 Bearing capacity factors

The following bearing capacity factor, N_{c}, in Equation (4) is recommended for vertical actions on a strip foundation with no embedment (plane strain conditions):

$$N_{\mathrm{c}} = \pi + 2 \tag{A.13}$$

The recommended bearing capacity factor is taken from Reference [32].

A.7.4.3 Bearing capacity correction factors

For the case with constant isotropic undrained shear strength with depth, the following bearing capacity correction factors in Equation (4) are recommended:

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

where

s_c is a shape correction factor $= 0{,}2(1 - 2i_c)(B'/L)$ (A.15)

d_c is a depth correction factor $= 0{,}3 \arctan(D_b / B')$ (A.16)

i_c is an inclined actions correction factor $= 0{,}5 - 0{,}5\sqrt{1 - \left[H_b / (A' c_u / \gamma_m)\right]}$ (A.17)

The recommended correction factors s_c and i_c are taken directly from Reference [32]. The recommended d_c taken from Reference [36] is slightly more conservative than specified by Reference [32]. B' and L are determined from Equations (A.5) to (A.9).

The relevancy of using the above depth factor d_c should be evaluated in each case. If the installation procedure and/or other foundation aspects, such as scour, do not allow for the required mobilization of shear stresses in the soil above foundation base level, it is recommended that d_c = 0,0.

In particular, it is recommended that d_c = 0 if the horizontal action leads to mobilization of significant passive earth pressure between sea floor and foundation base level.

A.7.5 Undrained bearing capacity — linearly increasing shear strength

A.7.5.1 Bearing capacity formulae

With regard to the bearing capacity equations presented in 7.5, the following bearing capacity factors and correction factors primarily come from References [32] to [36]. In general, these equations and factors should be used with care and their applicability should be checked in each case.

A.7.5.2 Bearing capacity correction factors

For the case with linearly increasing isotropic undrained shear strength with depth, the following correction factors F and K_c in Equation (5) are recommended:

F is taken from Figure A.1 as function of $\kappa B'/c_{u,0}$

$$K_c = 1 + s_c + d_c - i_c \tag{A.18}$$

where

$$s_c = s_{cv}(1 - 2i_c)(B'/L) \tag{A.19}$$

s_{cv} is taken from Table A.1 as function of $\kappa B'/c_{u,0}$

$$d_c = 0{,}3\left(\frac{c_{u,1}}{c_{u,2}}\right)\arctan\left(\frac{D_b}{B'}\right) \tag{A.20}$$

$$i_c = 0{,}5 - 0{,}5\sqrt{1 - \left[H_b / (A' c_{u,0} / \gamma_m)\right]} \tag{A.21}$$

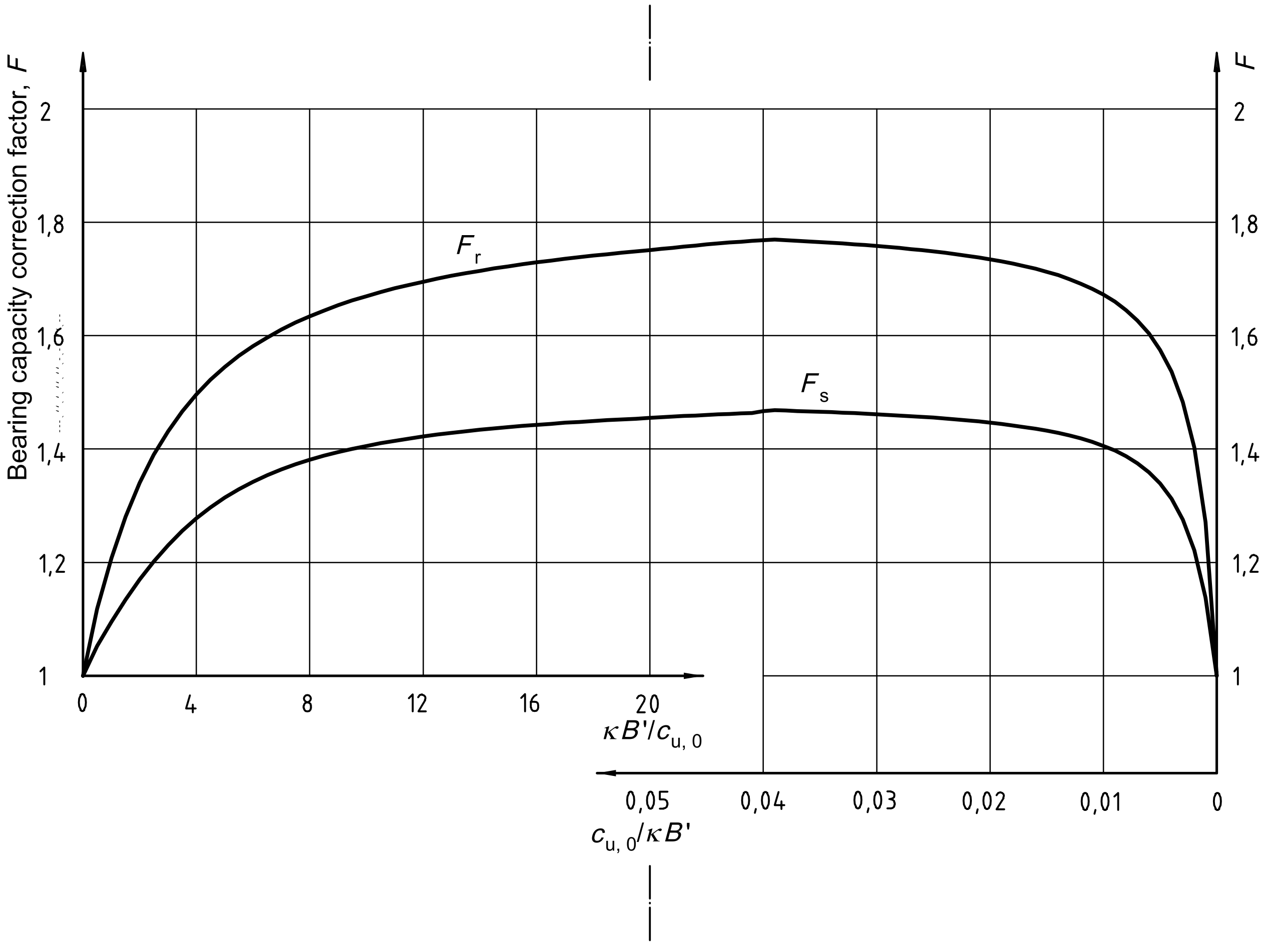
and

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

$c_{u,2}$ is the equivalent shear strength below base level, and

$$c_{u,2} = F\left(N_c c_{u,0} + \kappa B'/4\right)/N_c$$

B' and L are determined from Equations (A.5) to (A.9).



NOTE F_s, is for no friction at soil–foundation interface ("smooth" foundation), F_r, for friction equal to the shear strength of soil at the interface ("rough" foundation).

Figure A.1 — Bearing capacity correction factor F for linearly increasing isotropic undrained shear strength with depth

The correction factor F is taken from Reference [33]. The recommended d_c and i_c are based upon the correction factors for constant isotropic undrained shear strength, but modified to account for linearly increasing undrained shear strength with depth as given in Reference [36].

The shape factor, $s_{cv,}$ from Reference [35] for axial symmetry and pure vertical actions is assumed to be approximately valid also for an equivalent square foundation (B'/L) = 1; see Table A.1.

Table A.1 — Shape factor for circular or square foundation for pure vertical actions

$\kappa B'/c_{u,0}$	s_{cv}
0	0,20
2	0,00
4	− 0,05
6	− 0,07
8	− 0,09
10	− 0,10

The relevancy of using the above depth factor, d_c, should be evaluated in each case. If the installation procedure and/or other foundation aspects, such as scour, do not allow for the required mobilization of shear stresses in the soil above foundation base level, it is recommended that d_c = 0.

In particular, it is recommended that d_c = 0 if the horizontal action leads to mobilization of significant passive earth pressure between sea floor and foundation base level.

A.7.6 Drained bearing capacity

A.7.6.1 Bearing capacity formulae

With regard to the bearing capacity equations presented in 7.6, the following bearing capacity factors and correction factors primarily come from References [32] to [36]. In general, the equations and factors should be used with care. The relevancy should be checked in each case. The effective angle of internal friction, ϕ', should also be selected with care, see A.7.7.

A.7.6.2 Bearing capacity factors

The following bearing capacity factors for use in Equation (6) are recommended for pure vertical actions on a strip foundation with no embedment (plane strain conditions):

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

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

A.7.6.3 Bearing capacity correction factors

For the drained case, the following bearing capacity correction factors in Equation (6) are recommended:

$$K_q = s_q d_q i_q \quad \text{(A.24)}$$

$$K_\gamma = s_\gamma d_\gamma i_\gamma \quad \text{(A.25)}$$

where

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

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

$$i_q = \left\{1 - 0{,}5\left[H_b / \left(V_b + A'a\right)\right]\right\}^5 \tag{A.28}$$

$$s_\gamma = 1 - 0{,}4 i_\gamma (B'/L) \tag{A.29}$$

$$d_\gamma = 1 \tag{A.30}$$

$$i_\gamma = \left\{1 - 0{,}7\left[H_b / \left(V_b + A'a\right)\right]\right\}^5 \tag{A.31}$$

The relevancy of using the above depth factor d_q should be evaluated in each case. If the installation procedure and/or other foundation aspects, such as scour, do not allow for the required mobilization of shear stresses in the soil above foundation base level, it is recommended that d_q = 1,0.

In particular, it is recommended that d_q = 1,0 if the horizontal action leads to mobilization of significant passive earth pressure between sea floor and foundation base level.

A.7.7 Shear strength used in bearing capacity calculations

The value of ϕ' used in calculating the drained bearing capacity equations in A.7.6 should be relevant to plane strain conditions, which is generally 10 % higher than that measured in a triaxial compression test.

A.7.8 Settlements and displacements

Guidance on the calculation of immediate and consolidation settlements is given in standard soil mechanics textbooks [2] [3]. Guidance on the calculation of cyclic displacements may be obtained from ISO 19903.

It should be noted that immediate settlements and displacements can be significant at the maximum soil stress levels allowed in this document.

Consolidation and creep settlements increase with foundation dimensions and stress levels and decrease with overconsolidation ratio.

A.7.9 Dynamic behaviour

No guidance is offered.

A.7.10 Hydraulic stability

No guidance is offered.

A.7.11 Installation and removal

No guidance is offered.

A.7.12 Shallow foundations equipped with skirts

No guidance is offered.

A.7.13 Shallow foundations without skirts

No guidance is offered.

A.7.14 Installation effects

The use of relatively small installation vessels can result in foundation failure; vessel heave motions greater than crane pay-out speed can result in the structure having excessive velocity at impact with the sea floor and multiple set-downs. The result is often that the foundation is pulled up after first set-down, thus generating a pullout failure in the soil (reverse bearing capacity failure). The soil condition under the structure after such an event is close to a remoulded state and normal partial action and material factors can be too small to prevent foundation failure during final set-down.

Shallow foundations, such as those used for temporary support or for subsea structures, are often designed for limited environmental actions. Since the main actions are permanent and due to gravity, the higher material factor in 7.3 should be applied. If there is a risk for significant heave motions or impact on touchdown, a higher safety margin should be used. Experience indicates the total safety margin should be at least 2,0.

Annex B
(informative)

Carbonate soils

B.1 General

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

The comments below are focused primarily on carbonate silts and sands. Clay soils with varying proportions of carbonate content are common offshore, but there is little guidance as to how conventional design approaches for clay soils should be modified for different carbonate content. Local experience is important in making such assessments.

B.2 Characteristic features

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

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

This clause gives a general overview of the mechanical behaviour of carbonate soils. For a more detailed understanding of material characteristics, information can be found in References [37] to [64].

B.3 Properties

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

a) material composition, particularly carbonate content;

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

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

d) compressibility of the material;

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

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

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

B.4 Foundations

B.4.1 Driven piles

Several case histories have been reported that describe some of the unusual characteristics of foundations on carbonate soils and their often poor performance. It has been shown from numerous pile load tests that piles driven into weakly cemented and compressible carbonate sands and silts mobilize only a fraction of the capacity (as low as 15 %) predicted by conventional design and/or prediction methods for siliceous material. On the other hand, dense, strongly cemented carbonate deposits can be very competent foundation material. Unfortunately, the difficulty in obtaining high-quality samples and the lack of generalized design methods sometimes make it difficult to predict where problems can occur. With clays, care should be taken when the carbonate content exceeds 50 % and where no pile test data or local experience exist.

B.4.2 Other deep foundation alternatives

The current trend for deep foundations in carbonate sands and silts is a move away from driven piles. However, because of lower installation costs, driven piles still receive consideration for support of lightly loaded structures or where extensive local pile load test data and experience exists to substantiate the design premise. Furthermore, driven piles can be appropriate in moderately competent carbonate soils. At present, the preferred alternative to the driven pile is the drilled and grouted pile. Drilled and grouted piles mobilize significantly higher unit skin friction. The result is a substantial reduction in the required pile penetration compared with driven piles. Because of the high construction cost of drilled and grouted piles, an alternative driven and grouted pile system has received some attention in the recent past. This system has the potential to reduce installation costs while achieving comparable capacity. For any type of grouted pile, consideration should be given to the potential for reduction in shaft capacity due to cyclic actions, especially once slip has been initiated between pile and soil.

B.4.3 Shallow foundations

Shallow foundations are suitable for use on carbonate sediments, although any evaluation of such foundations should account for the important difference that exists with such material compared with silica sands or normal clays. Carbonate sands and silts generally have higher friction angles than silica sands and silts, but are more compressible, and these two factors influence bearing capacity in opposite ways. Carbonate sands and silts are also generally less permeable than equivalent silica material, leading to longer drainage times for a given size of foundation. The tendency for volume reduction on shearing, particularly under cyclic actions, combined with longer drainage times, leads to potential for bearing failure induced by soil liquefaction. It should also be noted that undrained cyclic strength of carbonate sands is generally lower than for most silica sands. The high compressibility of most carbonate sediments results in relatively large consolidation settlements, and can give rise to large settlements induced by cyclic actions. Shallow foundations are attractive for carbonate sediments that exhibit a significant degree of cementation, since they give high bearing capacities, good resistance to cyclic actions and low potential for settlements. However, layered profiles of variably cemented and uncemented sediments should be treated cautiously, with due account taken of the risks and consequences of a punch-through type of failure. Novel foundation systems such as those using suction assistance for penetration of skirts should be evaluated carefully on a case-by-case basis.

B.5 Assessment

To date, general design procedures for foundations in carbonate soils are not available. Acceptable design methods have evolved but remain highly site-specific and dependent on local experience. Stemming from some recent publications describing poor foundation performance in carbonate soils and the financial consequences of the remedial measures, there is a growing tendency to take a conservative approach to design in carbonate soils, even if the carbonate content in the sediment fraction is relatively low. This is not always warranted. As with other designs, the judgement of knowledgeable engineering remains a critical link in economic design of offshore foundations in carbonate soil environments.

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1) In preparation.

ICS 75.180.10

Price based on 34 pages