INTERNATIONAL STANDARD

ISO 19901-7

Second edition
2013-05-01

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

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

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

Partie 7: Systèmes de maintien en position des structures en mer flottantes et des unités mobiles en mer

Reference number
ISO 19901-7:2013(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 19901-7 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*.

This second edition cancels and replaces the first edition (ISO 19901-7:2005), which has been technically revised.

This second edition of ISO 19901-7 includes several major additions and changes, primarily to Annex A (informative). The largest change is the addition of detailed informative text incorporated directly from API RP 2SK on all types of anchor design. In the first edition of this International Standard, this material was previously addressed only by reference to API RP 2SK. Informative material has also been added from API RP 2SK regarding the analysis and mitigation of vortex-induced motions of large cylindrical hulls. Consequently, the normative text has been modified to remove reference to API RP 2SK and to cross-reference portions of the expanded informative annex.

The other significant change is the updating of guidance on polyester rope mooring design to conform to the provisions of the recent amendment to API RP 2SM. The changes include new definitions of stiffness, recognition of effective filter barriers, removal of the prohibition against the rope touching the sea floor, and more detail on minimum tension requirements, among others. Additionally, minor corrections were made to the text in 7.4.4 (Wind actions) and 8.3.4 (Riser considerations), and the terminology "most probable maximum" has been standardized throughout. Finally, the Norwegian clause of Annex B has been updated at the request of Norway, and a new Canadian clause has been added.

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*

The following parts are under preparation:

— *Part 8: Marine soil investigations*

ISO 19901 is one of a series of International 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 19904-1, *Petroleum and natural gas industries — Floating offshore structures — Part 1: Monohulls, semi-submersibles and spars*

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

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

— ISO 19905-3, *Petroleum and natural gas industries — Site-specific assessment of mobile offshore units — Part 3: Floating units*[1)]

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

1) Under preparation.

Introduction

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

This part of ISO 19901 was developed in response to the worldwide offshore industry's demand for a coherent and consistent definition of methodologies to analyse, design and evaluate stationkeeping systems used for floating production and/or storage platforms of various types (e.g. semi-submersibles, spar platforms, ship-shaped structures) and to assess site-specific applications of mobile offshore units (such as mobile offshore drilling units, pipelay units, construction units).

Stationkeeping is a generic term covering systems for keeping a floating structure, which is under the constant influence of external actions, on a pre-defined location and/or heading with limited excursions. Stationkeeping systems resist external actions by means of any combination of the following:

— mooring systems (e.g. spread moorings or single point moorings);

— dynamic positioning systems (generally consisting of thrusters).

The external actions generally consist of wind, wave, current and ice actions on the floating structure, mooring and/or risers.

Some background to, and guidance on, the use of this part of ISO 19901 is provided in informative Annex A. The clause numbering in Annex A is the same as in the normative text to facilitate cross-referencing.

Regional information, where available, is provided in informative Annex B.

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

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

1 Scope

This part of ISO 19901 specifies methodologies for

a) the design, analysis and evaluation of stationkeeping systems for floating structures used by the oil and gas industries to support

   1) production,

   2) storage,

   3) drilling, well intervention and production,

   4) production and storage,

   5) drilling, well intervention, production and storage, and

b) the assessment of stationkeeping systems for site-specific applications of mobile offshore units (e.g. mobile offshore drilling units, construction units, and pipelay units).

Most stationkeeping systems used with the class of floating structures covered by a) are termed "permanent mooring systems", for which this part of ISO 19901 is applicable to all aspects of the life cycle and includes requirements relating to the manufacture of mooring components, as well as considerations for in-service inspections. Most stationkeeping systems used with mobile offshore units, the class covered by b), are termed "mobile mooring systems". Throughout this part of ISO 19901, the term "floating structure", sometimes shortened to "structure", is used as a generic term to indicate any member of the two classes, a) and b).

This part of ISO 19901 is applicable to the following types of stationkeeping systems, which are either covered directly in this part of ISO 19901 or through reference to other guidelines:

— spread moorings (catenary, taut-line and semi-taut-line moorings);

— single point moorings, anchored by spread mooring arrangements;

— dynamic positioning systems;

— thruster-assisted moorings.

Descriptions of the characteristics and of typical components of these systems are given in Annex A.

The requirements of this part of ISO 19901 mainly address spread mooring systems and single point mooring systems with mooring lines composed of steel chain and wire rope. This part of ISO 19901 also provides guidance on the application of the methodology to synthetic fibre rope mooring systems, and includes additional requirements related to the unique properties of synthetic fibre ropes.

This part of ISO 19901 is applicable to single anchor leg moorings (SALMs) and other single point mooring systems (e.g. tower soft yoke systems) only to the extent to which the requirements are relevant.

This part of ISO 19901 is not applicable to the vertical moorings of tension leg platforms (TLPs).

The methodology described in this part of ISO 19901 identifies a set of coherent analysis tools that, combined with an understanding of the site-specific metocean conditions, the characteristics of the floating structure under consideration, and other factors, can be used to determine the adequacy of the stationkeeping system to meet the functional requirements of this part of ISO 19901.

NOTE For moorings deployed in ice-prone environments, additional requirements are given in ISO 19906.

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

ISO 19904-1, *Petroleum and natural gas industries — Floating offshore structures — Part 1: Monohulls, semi-submersibles and spars*

3 Terms and definitions

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

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

EXAMPLE An imposed deformation can be caused by fabrication tolerances, settlement, temperature change or moisture variation.

NOTE An earthquake typically generates imposed accelerations.

[ISO 19900:2002]

3.2
action effect
effect of actions on structural components

[ISO 19900:2002]

EXAMPLE Internal forces, moments, stresses, strains, rigid body motions or elastic deformations.

3.3
catenary mooring
mooring system where the restoring action is provided by the distributed weight of mooring lines

[ISO 19900:2002]

3.4
characteristic value
value assigned to a basic variable, an action or a resistance from which the design value can be found by the application of a partial factor

NOTE 1 The value usually has a prescribed probability of not being violated which, in the case of an action, will normally relate to a reference period.

NOTE 2 Adapted from ISO 19900:2002, definition 2.7.

3.5
design criteria
quantitative formulations that describe the conditions to be fulfilled for each limit state

[ISO 19900:2002]

3.6
design service life
assumed period for which a structure or a structural component is to be used for its intended purpose with anticipated maintenance, but without substantial repair being necessary

NOTE Adapted from ISO 19900:2002, definition 2.12.

3.7
design situation
set of physical conditions during a certain reference period for which the design will demonstrate that relevant limit states are not exceeded

NOTE Adapted from ISO 19900:2002, definition 2.13.

3.8
dynamic action
action that induces acceleration of a structure or a structural component of a magnitude sufficient to require specific consideration

3.9
dynamic positioning
DP
stationkeeping technique consisting primarily of a system of automatically controlled on-board thrusters, which generate appropriate thrust vectors to counter the mean and slowly varying induced actions

3.10
expected value
first-order statistical moment of the probability density function for the considered variable that, in the case of a time-dependent parameter, can be associated with a specific reference period

3.11
fit-for-purpose
fitness-for-purpose
meeting the intent of an International Standard although not meeting specific provisions of that International Standard in local areas, such that failure in these areas will not cause unacceptable risk to life-safety or the environment

[ISO 19900:2002]

3.12
floating structure
structure where the full weight is supported by buoyancy

[ISO 19900:2002]

NOTE The full weight includes lightship weight, mooring system pre-tension, riser pre-tension, operating weight, etc.

3.13
limit state
state beyond which the structure no longer fulfils the relevant design criteria

[ISO 19900:2002]

3.14
maintenance
set of activities performed during the operating life of a structure to ensure it is fit-for-purpose

3.15
minimum breaking strength
MBS
RCS certified strength of a chain, wire rope, fibre rope or accessories

3.16
mobile mooring system
mooring system, generally retrievable, intended for deployment at a specific location for a short-term operation, such as those for mobile offshore units (MOUs)

3.17
mobile offshore drilling unit
MODU
structure capable of engaging in drilling and well intervention operations for exploration or exploitation of subsea petroleum resources

3.18
mobile offshore unit
MOU
structure intended to be frequently relocated to perform a particular function

[ISO 19900:2002]

EXAMPLE Pipelaying vessel or barge, offshore construction structure, accommodation structure (floatel), service structure, or mobile offshore drilling units.

3.19
mooring component
general class of component used in the mooring of floating structures

EXAMPLE Chain, steel wire rope, synthetic fibre rope, clump weight, buoy, winch/windlass, fairlead or anchor.

3.20
owner
representative of the company or companies which own a development, who can be the operator on behalf of co-licensees

3.21
permanent mooring system
mooring system normally used to moor floating structures deployed for long-term operations, such as those for a floating production system (FPS)

3.22
proximity
closeness in distance

NOTE 1 Mooring systems are considered to be in proximity to a surface installation (or facility) if any part of the other installation lies within a contour described by the set of offsets coinciding with each line reaching 100 % MBS in the intact or redundancy check condition, whichever is larger.

NOTE 2 Mooring systems are considered to be in proximity to a sea floor installation (or facility) if any part of the other installation lies within a polygon formed by the anchor locations.

3.23
RCS
recognized classification society
member of the international association of classification societies (IACS), with recognized and relevant competence and experience in floating structures, and with established rules and procedures for classification/certification of installations used in petroleum-related activities

3.24
resistance
capacity of a structure, a component or a cross-section of a component to withstand action effects without exceeding a limit state

NOTE This definition is at variance with that specified in ISO 19900:2002.

3.25
return period
average period between occurrences of an event or of a particular value being exceeded

NOTE The offshore industry commonly uses a return period measured in years for environmental events. The return period is equal to the reciprocal of the annual probability of exceedance of the event.

[ISO 19901-1:2005]

3.26
riser
piping connecting the process facilities or drilling equipment on the floating structure with the subsea facilities or pipelines, or reservoir

NOTE 1 Possible functions include drilling and well intervention, production, injection, subsea systems control and export of produced fluids.

NOTE 2 Adapted from ISO 19900:2002, definition 2.29.

3.27
semi-submersible
floating structure normally consisting of a deck structure with a number of widely spaced, large cross-section, supporting columns connected to submerged pontoons

NOTE Pontoon/column geometry is usually chosen to minimize global motions in a broad range of wave frequencies.

3.28
serviceability
ability of a structure or structural component to perform adequately for normal functional use

3.29
significant value
statistical measure of a zero-mean random variable equal to twice the standard deviation of the variable

3.30
single point mooring
mooring system that allows the floating structure to which it is connected to vary its heading (weathervane)

EXAMPLE One example of a single point mooring is a turret mooring system where a number of mooring lines are attached to a turret, which includes bearings to allow the structure to rotate.

3.31
ship-shaped structure
monohull floating structure having a geometry similar to that of ocean-going ships

3.32
spar platform
deep-draught, small water-plane area floating structure

3.33
spread mooring
mooring system consisting of multiple mooring lines terminated at different locations on a floating structure, and extending outwards, providing an almost constant structure heading

3.34
stationkeeping system
system capable of limiting the excursions of a floating structure within prescribed limits

3.35
structural component
physically distinguishable part of a structure

[ISO 19900:2002]

3.36
structure
organized combination of connected components designed to withstand actions and provide adequate rigidity

[ISO 19900:2002]

3.37
taut-line mooring
mooring system where the restoring action is provided by elastic deformation of mooring lines

[ISO 19900:2002]

3.38
thruster-assisted mooring
stationkeeping system consisting of mooring lines and thrusters

3.39
verification
examination made to confirm that an activity, product, or service is in accordance with specified requirements

3.40
weathervaning
process by which a floating structure passively varies its heading in response to time-varying environmental actions

4 Symbols and abbreviated terms

4.1 Symbols

C coefficient (non-dimensional unless otherwise specified)

D annual fatigue damage, in years^{-1}

d diameter of the mooring line or component, in metres (m)

F direct action, in newtons (N), or a direct action per unit length, in newtons per metre, (N/m)

f frequency, in hertz (Hz)

K fatigue constant (non-dimensional unless otherwise specified)

k axial stiffness, in newtons per metre (N/m)

L design service life, in years

l length, in metres (m)

M mass, in kilograms (kg)

m inverse slope of T-N or S-N fatigue curves

N total number of (permissible) cycles

n number of cycles per annum, in year^{-1}

P probability of occurrence

S offset or motion, in metres (m)

S_R stress range, in megapascals (MPa)

s standard deviation

T tension force, in newtons (N); or non-dimensional tension ratio

t time, period or duration, in seconds (s)

v velocity, in metres per second (m/s)

W submerged weight, in newtons (N), or weight per unit length, in newtons per metre (N/m)

Γ gamma function

γ design safety factor

δ bandwidth parameter for the wave frequency

ε annual creep elongation, percent per year

σ ratio of the standard deviation of the tension variations around the mean tension to a reference breaking strength

ρ density, in kilograms per cubic metre (kg/m^3)

4.2 Abbreviated terms

ALS	accidental limit state
CALM	catenary anchor leg mooring
DP	dynamic positioning
FEA	finite element analysis
FLS	fatigue limit state
FMEA	failure modes and effects analysis
FPS	floating production system
FPSO	floating production, storage and offloading structure
FSO	floating storage and offloading structure
HMPE	high modulus polyethylene
IACS	International Association of Classification Societies
IMCA	International Marine Contractors Association
IMO	International Maritime Organization
LTM	long-term mooring
MBS	minimum breaking strength
MDS	mooring design states
MODU	mobile offshore drilling unit
MOU	mobile offshore unit
ORQ	oil rig quality
RAO	response amplitude operator
RCS	recognized classification society
ROV	remotely operated vehicle
SALM	single anchor leg mooring
SAW	submerged arc welding
SIM	structural integrity management
SLS	serviceability limit state
TAM	thruster-assisted mooring
TLP	tension leg platform
ULS	ultimate limit state
VIM	vortex-induced motion
VIV	vortex-induced vibration

5 Overall considerations

5.1 Functional requirements

The function of a stationkeeping system is to restrict the horizontal excursion of a floating structure within prescribed limits, as well as to provide means of active or passive directional control when the structure's orientation is important for safety or operational considerations.

The limiting criteria for excursions and orientation are generally established either by the owner of the floating structure or by direct derivation from design requirements including those related to

— safety of personnel,

— protection of the environment,

— stability and serviceability of the floating structure,

— serviceability of the topsides equipment,

— integrity and serviceability of drilling, production, export or other types of risers,

— access to and clearances with respect to nearby subsea or surface installations, and

— any other special positioning requirement.

Compliance of the stationkeeping system design with the requirements outlined above shall be established using the analysis methodologies given in Clauses 8 and 9, and the design criteria specified in Clause 10. The effects of external actions on the floating structure such as line tensions, structure offsets and anchor forces shall be evaluated for all relevant design situations, and shall be compared with the system and component resistances to ensure the existence of reserve strengths against mooring line breakage, offset exceedance, anchor slippage or other undesirable occurrences.

5.2 Safety requirements

Safety of life, environment and property shall be the main principles to be respected at all times through

— competent design or assessment, which ensures the ability of the floating structure and its stationkeeping system to withstand environmental and other external actions likely to occur during the design service lives of the structure and stationkeeping system, or duration of site-specific deployment of an MOU,

— definition of safe operating procedures so that risks of injuries to personnel are identified and minimized,

— identification and assessment of possible accidental events, as summarized in ISO 19900, and minimization of their consequences,

— performance of a risk assessment to ensure that possible malfunctions do not pose a danger to life or structure integrity, and

— compliance with all relevant regulations, see ISO 19904-1.

The implications of the above items shall be incorporated in the stationkeeping system design or assessment, and in the development of the operational philosophy.

5.3 Planning requirements

Planning shall be carried out before actual design or assessment is started in order to ensure that the stationkeeping system is able to perform its intended function according to 5.1. The initial planning shall include the determination of all conditions and criteria, in accordance with the general requirements and conditions specified in ISO 19900.

5.4 Inspection and maintenance requirements

The integrity of a stationkeeping system and its serviceability throughout the design service life are not only strongly dependent on a competent design, but also on the quality control exercised in manufacture, the supervision on-site, handling during transport and installation, and the manner in which the system is used and maintained.

At the planning stage, a philosophy for inspection and maintenance shall be developed and documented, to ensure full consistency with the design of the stationkeeping system and its components. A critical assessment shall be made of the ability to actually achieve the intended objectives through inspection and maintenance efforts. Relevant requirements related to inspection and maintenance requirements are given in Clause 12.

5.5 Analytical tools

Most of the analytical procedures and calculations described, specified and referenced in this part of ISO 19901 are commonly performed with the assistance of computer-aided engineering tools. Many of these consist of commercially available, widely used software suites which, when employed by experienced and well-trained users, may be considered *de facto* industry standards. For these software systems, the original author is expected to have performed adequate validation and verification, and to maintain evidence thereof.

In other cases, particularly in technological areas undergoing rapid evolution, innovative analytical approaches and techniques are often embedded in original, proprietary software solutions. In such cases, the developer is expected to validate the adequacy of the results by, for instance, comparison with test data or field measurements.

In any case, the designer shall document that the tools and modelling protocols used in the design and analysis activities have been shown to provide results considered acceptable in terms of consistency and accuracy when compared to test data, field measurements, or to the results of other similar tools.

6 Design requirements

6.1 Exposure levels

6.1.1 General

Like all offshore structures, floating structures vary in size, complexity, mission, performance requirements, manning levels, criticality to the asset development strategy, possible hazards, etc. In order to define appropriate design situations and design criteria for a particular structure, the concept of exposure levels was introduced.

According to this philosophy, an offshore structure at a particular location is characterized by a specific exposure level. Associated with each exposure level are appropriate design situations and design criteria for the structure's intended service.

Exposure levels are determined considering in combination life safety and consequences of failure for a given structure. Life safety is a direct function of the structure's expected manning levels during the environmental design situation. Consequences of failure are mainly related to the potential risk to life of personnel brought in to respond to any incident, the potential risk of environmental damage and the potential risk of economic losses.

These concepts and definitions apply to the design of the class of floating structures covered under a) but not to those of b) (mobile offshore units) as given in the Scope of this part of ISO 19901.

The definition of the exposure level for floating structures is given in ISO 19904-1.

6.1.2 Exposure levels for stationkeeping systems

The exposure level assigned to a permanent stationkeeping system shall be no less onerous than the exposure level of the floating structure to which it is connected.

6.2 Limit states

6.2.1 General

The general principles on which design requirements for offshore structures are based are documented in ISO 19900. These state that design verification of a system and its components shall be performed with reference to a specified set of limit states beyond which the structure or the system no longer satisfies the requirements of Clause 5.

For each limit state, appropriate design situations shall be defined, calculation models shall be established, design criteria shall be defined, and adequate procedures shall be followed to verify compliance with design requirements.

6.2.2 Limit states for stationkeeping systems

ISO 19900 identifies four categories of limit states:

— ultimate limit states (ULS);

— serviceability limit states (SLS);

— fatigue limit states (FLS);

— accidental limit states (ALS).

6.3 Defining design situations

The definition of specific design situations for the stationkeeping system is the responsibility of the owner in accordance with the requirements of a regulatory authority where one exists. Aspects to be considered in determining design situations include

— service requirements for the stationkeeping system,

— design service life,

— hazards (e.g. accidental events) to which the stationkeeping system and the connected floating structure can be exposed during its design service life,

— potential consequences of partial or complete stationkeeping system failure, and

— nature and severity of environmental conditions to be expected during the design service life.

6.4 Design situations

6.4.1 General

Provisions related to the consideration of environmental conditions and their application are given in ISO 19900, and these shall be complied with in conjunction with the further requirements of ISO 19901-1 and those of this part of ISO 19901.

Design situations include all the service and operational requirements resulting from the intended use of the floating structure and the environmental conditions that could affect the stationkeeping system according to ISO 19900.

In particular, an environmental design situation consists of a set of actions induced by waves, wind, current and ice (if any) on the floating structure, on the risers and on the mooring system, as applicable, and is characterized by a given return period for one or more environmental variables or for a contour of environmental variables.

In the absence of site-specific information on environmental conditions, ISO 19901-1 gives an approximate indication of the extreme conditions for certain geographic areas. These values are meant to be indicative only (e.g. for conceptual design studies). The owner shall review the validity of these values and, if necessary, use site-specific data for the final design instead of those provided therein.

Criteria to be met by the design can be directly related to the specific formulation of the design situations. In this case, design situations, calculation process and design criteria are interrelated and should not be separated from one another.

For permanent moorings, the design situations detailed below apply to mooring systems for floating structures with an L1 exposure level — see ISO 19904-1 for the definition of exposure levels.

6.4.2 Design situations for ULS

6.4.2.1 General

The parameters specifying the environmental design situation should be developed from the environmental information as described in 6.4.2.2 for permanent moorings and 6.4.2.3 for mobile moorings.

6.4.2.2 Permanent moorings

6.4.2.2.1 General

For permanent moorings, the return periods of the parameters that characterize environmental design situations should be several times the design service life of the stationkeeping system.

Parameters characterizing ULS environmental design situations in this part of ISO 19901 shall be based on a return period of 100 years, except as specified below.

6.4.2.2.2 Permanent moorings with a short design service life

When the design service life of the mooring system is substantially lower than 20 years, parameters characterizing design situations with return periods shorter than 100 years may be adopted. In such cases, the return period shall be determined through a risk assessment, taking into account the possible consequences of mooring system failure.

6.4.2.2.3 Permanent moorings designed for disconnection

Mooring systems can be designed to disconnect from the floating structure in advance of certain adverse environmental events, e.g. iceberg impact or hurricanes. The actions on the floating structure due to such adverse events may be ignored at the ULS, provided that either

a) an appropriate ALS is satisfied, consisting of the adverse environmental event combined with a failure to safely disconnect, or

b) the joint probability of occurrence of the adverse environmental event combined with the failure to disconnect is less than 10^{-4} per annum.

Furthermore, the mooring system design shall also be verified for a ULS design situation consisting of the actions of waves, wind, current and ice (if any) on the mooring system alone (i.e. without the floating structure).

6.4.2.2.4 Permanent moorings in proximity to other installations

When permanent moorings are in proximity to other installations, consideration should be given to increasing the return period of the design situation parameters for the moorings, in order to account for the possible consequences of contact with surface, mid-depth or sea floor infrastructures or installations.

6.4.2.2.5 Permanent moorings redundancy check condition

Permanent moorings shall be designed with sufficient redundancy so as to withstand the appropriate design situations (see 6.4.2.2.1 to 6.4.2.2.4) even after the loss of any one mooring line or any one or more thrusters as appropriately assessed by the failure modes and effects analysis (FMEA). This can be the consequence of accidental breakage, planned maintenance or local failure.

6.4.2.3 Mobile moorings

6.4.2.3.1 Mobile moorings for structures not in proximity to other installations

Environmental design situations for mobile moorings not in proximity to other installations shall be characterized by parameters with a return period of at least 5 years.

A risk analysis examining various mooring failure scenarios shall be conducted to evaluate the consequence of a mooring failure to demonstrate that the risk posed by loss of station is acceptable.

In tropical cyclone areas, the wind speed should be ≥ 30 m/s (1 min average at 10 m height).

6.4.2.3.2 Mobile moorings for structures in proximity to other installations

When mobile moorings are in proximity to other installations, the environmental design situation shall be characterized by parameters with return periods of at least 10 years, to account for the possible consequences of contact with surface, mid-depth or sea floor infrastructures or installations.

6.4.2.3.3 Mobile moorings redundancy check condition

Mobile moorings shall be designed with sufficient redundancy to be able to withstand the appropriate design situations (see 6.4.2.3.1 and 6.4.2.3.2) even after the loss of any one mooring line or any one or more thrusters as appropriately assessed by the FMEA. This can be the consequence of accidental breakage, planned maintenance or local failure.

6.4.3 Design situations for SLS

Design situations for SLS shall be defined by the owner either directly or with reference to a specific percentage of time on station when the structure is required to fulfil its intended mission.

6.4.4 Design situations for FLS

Design situations for FLS shall consist of a set of environmental states described by

— wave period, height and direction,

— wind speed and direction,

— current speed, profile and direction (see 8.3.5.5 for VIM-related issues), and

— the frequency of occurrence of each environmental state,

adequately representing the long-term statistics of the local environment, see Clause 9.

6.4.5 Design situations for ALS

No ALS events are specified for stationkeeping systems in this part of ISO 19901, except those of 6.4.2.2.3, to be applied as appropriate. However, consideration should be given to performing a site-specific assessment to establish the nature and probability of possible ALS design situations, e.g. iceberg impact. Accidental events with return periods in excess of 10 000 years may be neglected.

7 Actions

7.1 General

This clause addresses the main actions to be considered in the design of stationkeeping systems. As floating structure motions and offsets are generally the main contributors to stationkeeping system design, it also includes actions applied only to the floating structure, which result in indirect actions applied to the stationkeeping system.

7.2 Site-specific data requirements

7.2.1 Data collection and analysis

The interaction of environmental phenomena, such as wind, waves, current and tide, is site-specific. The joint probability distribution describing these actions should be considered in the development of the analytical and statistical models intended to predict actions and their effects.

When collecting data where joint probabilities are to be considered, care shall be exercised to preserve the appropriate information. Of particular importance are the wind/wave, wave height/wave period, and wave/current relationships and their absolute and relative directionality characteristics.

There are areas governed by special metocean phenomena that are not well represented by parameters with typical return period statistics. For example, some areas of generally mild climate can be subject to “sudden storms” such as squalls, and other areas can be subject to occasional very high currents. In these cases, the special occurrences shall be considered in determining the relevant environmental conditions.

The environmental action characteristics defining design situations for mobile moorings shall be determined from annual statistics. However, if the operating season is well defined, and seasonal environmental data are sufficient to provide meaningful statistics, these characteristics may be determined from seasonal statistics.

For fatigue analyses, wave, wind and current data shall be collected sufficient to satisfy 6.4.4.

Further information on data description and the data gathering procedures can be found in ISO 19901-1.

7.2.2 Water depth

The design water depth for the mooring system at each anchor location shall account for sea level variations due to tides and storm surges.

7.2.3 Soil and sea floor conditions

Sea floor soil conditions shall be investigated for the intended site to provide data for the anchoring system design. The sea floor slope shall be properly accounted for in the mooring analysis, when relevant. For permanent moorings a bottom survey shall be performed.

7.2.4 Wave statistics

The wave height versus wave period and wave direction relationships for the design situations shall be accurately determined from oceanographic data for the area of operation.

7.2.5 Wind statistics

Wind speed and direction data shall be accurately determined from the oceanographic data for the site under consideration.

7.2.6 Current profile

Current speed and direction data shall be collected from the oceanographic data for the area of operation. Appropriate consideration of current profiles on mooring lines/risers is often necessary for more detailed accurate analytical representations.

7.2.7 Atmospheric icing

Where applicable, an assessment shall be made of atmospheric icing, including sea spray icing. Increased wind area due to superstructure icing shall be considered in the calculations of wind actions on a floating structure.

7.2.8 Marine growth

The type and accumulation rate of marine growth at the design site can affect mass, weight, hydrodynamic diameters, and drag coefficients of floating structure members and mooring lines. This shall be taken into consideration for permanent mooring systems not subject to any regular marine growth removal.

7.3 Environmental actions on mooring lines

7.3.1 General

In this subclause, methods for evaluating direct environmental actions on mooring lines are summarized. Mooring lines are typically modelled as slender cylindrical members. Direct wave actions on mooring lines may generally be neglected.

7.3.2 Current-induced actions

The effect of current actions on mooring lines on the overall mooring design shall be evaluated. Current actions are likely to be particularly important for deepwater locations with high currents. Actions on mooring lines due to currents can be calculated from Equation (1):

$$F = 1/2 \rho_{\mathrm{w}} C_{\mathrm{d}} \cdot d \cdot v^2 \tag{1}$$

where

F is the force per unit length normal to the local mooring line, in newtons per metre (N/m);

ρ_w is the density of the seawater, in kilograms per cubic metre (kg/m^3);

C_d is the drag coefficient, see relevant RCS rules;

d is the nominal diameter of the mooring line, in metres (m);

v is the component of current velocity normal to the local mooring line, in metres per second (m/s).

Where there are high currents, drag coefficients should be adjusted for the presence of vortex-induced vibrations.

7.3.3 Ice-induced actions

In ice-prone areas, ice-induced actions on the mooring system shall be in accordance with ISO 19906.

7.3.4 Vortex-induced vibrations of mooring lines

For smooth cylindrical mooring lines the possibility of vortex-induced vibrations (VIV), in particular the effect on drag coefficient, should be considered.

7.4 Indirect actions

7.4.1 General

Floating structure offset and motions are the main indirect actions on a stationkeeping system. The details of the calculation of environmental actions on the floating structure are presented in ISO 19904-1.

7.4.2 Frequency ranges

For the purpose of assessing their effects, and sometimes considering their relative influence, environmental actions on floating structures can be categorized as follows according to their frequency range.

— Steady actions, such as wind, current and wave drift, which are constant in magnitude and direction for the duration of interest.

— Low-frequency cyclic actions (often referred to as slow drift), with characteristic periods between 1 min and 10 min, which typically induce dynamic excitation of floating structures at their natural periods in surge, sway and yaw. In spar platforms these actions can also induce dynamic excitation at the pitch and roll natural periods;

— Wave-frequency cyclic actions with characteristic periods ranging from 3 s to 30 s.

7.4.3 Wave-induced actions

Wave-induced actions (steady, low-frequency and wave-frequency) shall be determined by relevant analytical or empirical methods or model testing, with appropriate consideration of water depth effects. Relevant analytical/empirical methods include diffraction and radiation theory, and slender member hydrodynamics.

As the wave period can significantly affect slow drift motions, a range of wave periods should be investigated in accordance with A.7.2.4.

Interaction between current and waves shall be examined. This includes the change in intrinsic wave frequencies to apparent wave frequencies (see ISO 19901-1) as well as changes in the magnitudes of wave-induced actions.

7.4.4 Wind-induced actions

Drag coefficients shall be determined by means of wind tunnel tests and/or empirical analysis tools, see ISO 19904-1. The constant wind action from a particular direction should incorporate all three horizontal plane components (F_x, F_y, M_z) using force coefficients derived from model tests or calculation. For spars, wind pitch and roll moments are also important as the mean pitch and roll angle will also have an impact on the surge, sway and yaw wind action coefficients.

Two analytical approaches are generally used to represent steady and low-frequency wind actions (see ISO 19904-1):

a) the wind is treated as constant in direction and speed, which is taken as the 1 min average;

b) the wind is modelled by a steady component, based on the 1 h average velocity, plus a time-varying component calculated from a suitable empirical wind gust spectrum, see ISO 19901-1.

The design wind speed should refer to an elevation of 10 m above still water level.

For ULS design of permanent moorings, approach b) shall be taken; approach a) may also be used provided it can be shown to be more conservative. However, for FLS, b) shall be used because the effect of time-varying wind actions on the floating structure can contribute to the magnitude of the low-frequency tension cycles.

For mobile moorings, either a) or b) may be used.

For sites affected by tropical squalls, the concept of a wind spectrum is not applicable. The analysis of corresponding wind-induced actions and effects should be performed using an appropriate specification of a wind speed time history.

7.4.5 Current-induced actions

Current-induced actions (steady and low-frequency) on large body floating structures shall be determined by means of model tests and/or empirical analysis tools. Current-induced actions on slender members, including risers, may be determined using Equation (1).

7.4.6 Directional distribution

The floating structure offsets and motions to be used in the stationkeeping system design shall be evaluated for the most unfavourable combinations of wind directions, wave directions and current directions, consistent with the site-specific metocean characteristics. The ability of the floating structure to change heading in response to changing environmental conditions may be taken into account.

7.4.7 Vortex-induced motions of floating structures

Floating structures consisting of large diameter cylindrical components such as spars, semi-submersibles, and TLPs can experience low-frequency motions due to vortex shedding in the presence of currents. These vortex-induced motions (VIM) are most prominent for spars where most of the industry experience has been acquired. Nevertheless, multi-column floating structures such as semi-submersibles and TLPs can also experience VIM and this effect should be taken into account in their design.

VIM has three primary effects on the mooring design:

a) the average in-line drag coefficient is higher than the value it would have in the absence of VIM;

b) the low frequency VIM motions can be significant in terms of the total floating structure response;

c) the VIM motions cause additional low frequency oscillating mooring line tensions.

These effects should be taken into account for strength and fatigue design of the mooring system. The occurrence of the Loop Current and associated eddies in the Gulf of Mexico make consideration of VIM particularly important for this geographic area. For example, unlike other extreme events, e.g. winter storms and hurricanes, the Loop Current and associated eddies can affect a particular site for an extended period of time and can make a significant contribution to the fatigue damage of mooring components.

NOTE Waves transverse to a current can amplify VIM motions due to current only, dependent on wave frequency, so wave effects should be considered in a VIM analysis, if applicable.

8 Mooring analysis

8.1 Basic considerations

8.1.1 Introduction

Mooring analysis shall be performed to predict extreme values of action effects (maximum or minimum, as applicable) such as floating structure offsets, line tensions and anchor forces under environmental and other actions (riser actions, tandem mooring actions, etc.). The extreme action effect values shall be checked against the design criteria given in Clause 10.

Mooring line tensions are often manually adjusted for operational reasons and/or in advance of foreseeable environmental events. For the analysis of ULS design situations, the modelling of the stationkeeping system and floating structure configuration shall reflect such adjustments. However, the modelling of active adjustment of line tensions during the analysis of design situations shall not be taken into account.

The value of the mooring line diameter used in the mooring line analysis shall be the nominal value (i.e. with no deduction for corrosion or wear), unless noted otherwise below.

8.1.2 Mooring analysis conditions

8.1.2.1 General

Mooring analysis shall be carried out in accordance with 8.8 for the intact, redundancy check and transient conditions as defined below. Descriptions of analysis conditions for thruster-assisted moorings are given in 8.9.2.

Installation tolerances on anchor placement and line length should be taken into account in the mooring system design.

8.1.2.2 Intact condition

This is the condition in which all mooring lines are intact and all thrusters, if any, are working.

8.1.2.3 Redundancy check condition

This is the condition in which the structure has a new mean position after a single mooring line breakage or a failure of one or more thrusters as appropriately assessed by the FMEA.

A risk assessment of the stationkeeping system shall be performed if the structure is in the redundancy check condition.

8.1.2.4 Transient condition

This is the condition in which the structure undergoes transient motions between the intact and redundancy check conditions, including the possibility of overshooting, as a result of a single mooring line breakage or a failure of one or more thrusters as appropriately assessed by the FMEA.

8.1.2.5 Recommended analysis methods and conditions

The analysis methods (see 8.3.5) recommended to be used, depending on the conditions to be analysed and the limit states to be satisfied, are given in Table 1.

Table 1 — Recommended analysis methods and conditions

Type of mooring	Limit state	Conditions to be analysed	Analysis method
Permanent mooring	ULS	Intact/Redundancy check	Dynamic
		Transient [a]	Quasi-static or Dynamic
	FLS	Intact	Dynamic
	SLS	No guidance given	No guidance given
Mobile mooring	ULS	Intact/Redundancy check	Quasi-static or Dynamic
		Transient [a] [b]	Quasi-static or Dynamic
	FLS	Not required	Not applicable
	SLS	No guidance given	No guidance given

[a] Applicable only if another installation is in proximity to the mooring.

[b] Applicable for MODUs drilling in deepwater where excessive transient motions can cause stroke-out of the riser slip joint.

8.2 Floating structure offset

8.2.1 General

The following definitions of floating structure offset apply to any chosen reference point on the structure.

8.2.2 Mean offset

The mean offset is defined by the structure's rigid body displacement due to the combination of the steady components of wind, wave, current and other external actions. While, in general, only the three components in the horizontal plane (in the surge, sway and yaw directions) are taken into account, the other three components (in the heave, roll and pitch directions) can also be significant, in particular for small water-plane area structures.

8.2.3 Extreme values of offset

When the frequency-domain approach, 8.3.1.2, is used to compute the structure response, the extreme values of the offset are defined as the mean offset plus or minus the maximum estimated value of the time-varying excursion due to combined wave-frequency and low-frequency structure motions. Extreme values of the offset shall be also calculated relative to each anchor location for the purpose of tension calculation.

The extreme values of the offset, S_{max} and S_{min} can be determined by Equations (2) and (3).

$$S_{\mathrm{max}} = S_{\mathrm{mean}} + \mathrm{MAX}\,(S_{\mathrm{dyn1}}, S_{\mathrm{dyn2}}) \tag{2}$$

$$S_{\mathrm{min}} = S_{\mathrm{mean}} - \mathrm{MAX}\,(S_{\mathrm{dyn1}}, S_{\mathrm{dyn2}}) \tag{3}$$

$$S_{\mathrm{dyn1}} = S_{\mathrm{lfmax}} + S_{\mathrm{wfsig}} \tag{4}$$

$$S_{\mathrm{dyn2}} = S_{\mathrm{wfmax}} + S_{\mathrm{lfsig}} \tag{5}$$

where

MAX is the larger of the absolute values of the terms in parentheses;

S_{max} is the maximum structure offset, in metres (m);

S_{min} is the minimum structure offset, in metres (m);

S_{mean} is the mean structure offset, in metres (m);

S_{lfmax} is the most probable maximum value of low-frequency motion, in metres (m);

S_{lfsig} is the significant value of low-frequency motion, in metres (m);

S_{wfmax} is the most probable maximum value of wave-frequency motion, in metres (m);

S_{wfsig} is the significant value of wave-frequency motion, in metres (m).

Definitions of most probable maximum and significant values are given in 8.3.2.

The combined offset for different degrees-of-freedom (e.g. surge and sway) should be defined as the vector sum of the individual components of the offset.

Alternatives to this approach are the time-domain approach, 8.3.1.3, and the combined time-and-frequency-domain approach, 8.3.1.4, which involve statistical processing of simulated time histories to yield extreme values of the offsets.

The above subclause applies to the intact and redundancy check conditions. For the transient condition, extreme values of the offset are defined in 8.10.2.

8.3 Floating structure response

8.3.1 Analysis methods

8.3.1.1 General

The three methods generally used to compute floating structure response are

— the frequency-domain approach,

— the time-domain approach,

— the combined time- and frequency-domain approach.

These methods involve different degrees of approximation and are affected by different limitations, and therefore do not necessarily yield consistent results. If verification of the approach selected for the mooring design is required, model test data or an alternative approach should be used.

Currently, as there is no established analytical method for determining the motion response of floating structures undergoing VIM, the industry relies mainly on model testing. However, limited full-scale data are available to confirm the scalability of model testing. If current loadings and VIM are determined to be a design driver, it is usual practice to perform well planned model tests to determine motion amplitudes and drag coefficients for use in mooring design.

8.3.1.2 Frequency-domain approach

In this approach, the general equations of motion describing the response of the structure are decoupled and analysed separately for mean, low- and wave-frequency responses. Mean responses are calculated from static equilibrium between the steady environmental actions and the mooring system's restoring force. Wave-frequency and low-frequency structure motions are calculated from the frequency-domain approach which yields motion response statistics. Extreme values, such as significant and most probable maximum responses, are then evaluated based on peak probability density distributions, see 8.3.2. Finally, the most probable maximum values and significant values of wave-frequency and low-frequency responses are combined with the mean response, Equations (2) and (3), to yield the combined most probable maximum response for a specified storm duration.

To perform a frequency-domain analysis of a weathervaning structure, the structure's heading shall be fixed. The fixed design headings at which the mooring system responses are calculated shall be determined, taking into consideration the mean equilibrium heading and low-frequency yaw motions. Normally more than one fixed heading shall be considered to ensure the largest maximum has been identified, see 8.8.2.

8.3.1.3 Time-domain approach

In this approach, the general equations of motion describing the combined mean, low-frequency and wave-frequency responses of the floating structure are solved in the time domain. The forcing functions include the mean, low-frequency and wave-frequency actions due to wave, wind, current and thrusters, if any. The equations describing the floating structure, mooring lines, risers and thruster behaviour and their interactions are all included in the time-domain simulation. Time histories of all main response parameters (structure displacements, mooring line tensions, anchor forces, etc.) are obtained from the simulation, and the resulting time histories are then processed statistically to yield extreme values. The time-domain simulation should be long enough to yield stable statistical values.

8.3.1.4 Combined time-domain and frequency-domain approach

To reduce the complexity and computational effort associated with the fully coupled time-domain simulation, a combined time- and frequency-domain approach is often employed. Time- and frequency-domain solutions for mean responses, wave and low-frequency responses can be combined in different ways. In a typical approach, the mean and low-frequency responses (structure displacements, mooring line tensions, anchor forces, etc.) are simulated in the time domain while the wave-frequency responses are solved separately in the frequency domain. The frequency-domain solution for wave-frequency response is processed to yield either statistical extreme values or time histories, which are then superimposed on the mean and low-frequency responses.

8.3.2 Extreme value statistics

For time-domain analysis, relevant extreme values can be determined in accordance with A.8.3.1.3.

In frequency-domain analysis, for phenomena that can be represented by a narrow-banded Gaussian process, extreme value statistics for the time-varying component of the phenomenon can be calculated from the standard deviation of the relevant response spectrum as follows (Rayleigh distribution):

$$V_{\text{sig}} = \pm 2s \quad (6)$$

$$E_{\text{max}} = +s\sqrt{2\log_e(t/t_z)} \quad (7)$$

$$E_{\text{min}} = -s\sqrt{2\log_e(t/t_z)} \quad (8)$$

where

V_{sig} is the significant value;

E_{max} is the most probable maximum value;

E_{min} is the most probable minimum value;

s is the standard deviation;

t is the duration of the design situation, in units of time; a minimum of 3 h should be specified for the duration of the design situation;

t_z is the average zero up-crossing period of the motions, in units of time, where, for low-frequency motions, t_z can be taken as the natural period for the appropriate degree-of-freedom of the combined structure/riser/mooring system t_n, which can be estimated by

$$t_n = 2\pi\sqrt{M/k} \tag{9}$$

M is the system mass including added mass, in kilograms, (kg);

k is the system stiffness for the appropriate degree-of-freedom at the structure's mean position, in newtons per metre (N/m).

The most-probable maximum tension, to be compared with the line tension limits in 10.2, can be obtained from Equation (7) by adding the value of the mean tension.

NOTE These formulations can under-predict the most probable maximum value in cases of non-linear wide band response.

8.3.3 Low-frequency damping

Special attention should be given to low-frequency damping. Low-frequency motion of a moored structure is narrow-banded in frequency since it is dominated by the resonant response at the natural frequency of the moored structure. The motion amplitude is highly dependent on the stiffness of the mooring system and the damping. There is a substantial degree of uncertainty in the estimation of low-frequency damping of which there are four main sources:

— viscous damping of the structure;

— wave drift damping of the structure;

— mooring system damping;

— riser system damping.

8.3.4 Riser considerations

The riser system (if any) interacts with the floating structure and the mooring in several aspects. Wave and current actions on the risers increase the environmental actions to be resisted by the mooring, while the riser system stiffness provides assistance to the mooring. Furthermore, damping from the riser system decreases the low-frequency motions and in turn reduces the mooring tensions. The net result of these effects depends on a number of factors such as type and number of risers, and water depth. Mooring design should take into consideration the riser actions, stiffness and damping. The risers effects may be ignored if doing so results in a more conservative mooring design.

If a riser system is designed for different numbers of risers during its design service life (e.g. associated with later tie-in) all the different configurations shall be investigated.

8.3.5 Vortex-induced motion considerations

8.3.5.1 General

Unlike other resonant responses, the amplitude of VIM is bounded. Transverse motion behaviour is usually characterized by the non-dimensional ratio of the motion amplitude, a, to the body's diameter, d, (a/d). The largest single amplitude transverse motion observed on bluff bodies is on the order of $a/d = 1$. Helical strakes are commonly used on spars (and risers) to reduce the motion amplitude. Strakes can be very effective in eliminating VIM; however their effectiveness on spars depends on various factors, such as the exact layout and size of the strakes, appurtenances, and current profiles.

Special issues for VIM design and analysis include but are not limited to the following.

a) There is no established analytical tool for the prediction of VIM. Currently VIM design criteria are typically obtained from model testing. Model testing practices should be validated with field measurement data, which are quite limited.

b) VIM is affected by the natural periods of the combined floating structure and stationkeeping system, current velocity, direction, profile, hull geometry and appurtenances.

c) The duration of current resulting in VIM can be much longer than peak storm duration.

d) Model tests can only model certain parameters while approximating others, hence care should be exercised in the interpretation and use of model test data.

e) Where VIM results in large tension cycles at high mean load, fatigue life can be short for mooring components with low fatigue resistance such as chain.

f) The calibration of the factors of safety for mooring design does not include the VIM condition and the uncertainties associated with VIM. Consequently, sensitivity checks as described in A.8.3.5.1 are recommended.

Because of the above issues, it is important to address VIM conservatively in the mooring design stage. This can be achieved through the following measures:

— establishing design criteria that recognize the uncertainties in VIM behaviour, for example checking sensitivity cases in addition to the base case and checking field measurement data as well as model test data;

— conducting fatigue analysis for the 100-year VIM response condition in addition to long term fatigue analysis;

— selecting mooring hardware and system design characteristics that can better tolerate or mitigate VIM.

8.3.5.2 Design criteria for VIM strength analysis

The first step in strength design is to establish suitable VIM design criteria. VIM-related design parameters for mooring strength design include:

— in-line and transverse VIM response amplitude (a/d) as a function of reduced velocity (V_r);

— drag coefficient as a function of VIM response amplitude;

— definition of ranges for V_r;

— VIM response trajectory or envelope.

These criteria are generally based on a combination of project specific model test data and previous VIM design experience. Depending on the approach taken, there can be varying levels of uncertainty in the VIM criteria specified for a particular application. Criteria should be developed for a base case (the best estimate) and for some sensitivity cases. Tension safety factors for intact and damaged conditions should be met for the base case. Sensitivity cases should be used to check the robustness of the mooring system design, with the intent of confirming that the risk of mooring failure is at an acceptable level even in the event that estimates of certain influential parameters such as mooring stiffness, current velocity, drag coefficient, lock-in definition, or VIM amplitude, are inaccurate. One of the important roles of the sensitivity check is to determine if, with some limited changes in critical parameters, the system would enter a VIM lock-in regime that would not be apparent for the base-case design criteria alone.

8.3.5.3 VIM strength analysis method

Most mooring analysis software is not generally designed to handle VIM analysis, and therefore the simplified analysis procedure described in the informative annex is typically used.

8.3.5.4 Basic considerations for VIM fatigue analysis

VIM-induced mooring tensions are of a cyclic nature, and contribute to the mooring system fatigue damage. The following factors should be considered when assessing fatigue due to VIM:

a) For the calculation of the number of tension cycles, use should be made of the VIM period in the offset position, corresponding to the specific current bin under consideration. This period can vary with current direction and magnitude and is generally different from the still water natural period.

b) In addition to a long-term fatigue damage evaluation, a fatigue analysis for the 100-year VIM fatigue event (or other single worst-case event as noted in 8.3.5.5) is also recommended.

c) Mooring systems experiencing a high mean tension and large tension variation can stress the chain beyond the elastic region, where fatigue test data are not available. To ensure sufficient fatigue life, mooring systems should be designed to avoid this situation.

d) Fatigue damage of chain at the fairlead requires special attention since additional bending stress is imposed on the chain in this region, and chain typically has the lowest fatigue life of all the components in the mooring system.

e) Sensitivity cases, similar to those used in the strength analysis, should be considered to account for uncertainty in the VIM predictions.

8.3.5.5 VIM fatigue analysis for long term and single extreme events

For long-term fatigue analysis under VIM conditions, current events can be represented by a number of discrete current bins, with each current bin consisting of a reference direction, a reference current velocity and profile, associated wave and wind conditions, and probability of occurrence. Fatigue damage for each current bin is evaluated, and the fatigue damage due to VIM is combined with the fatigue damage due to wind and waves to yield total fatigue damage.

However, studies indicate that considerable fatigue damage can be caused even by a single extreme VIM event. Consequently, in addition to the long-term fatigue damage evaluation, a fatigue analysis of the 100-year VIM event should be considered. Since VIM response is largely dependent on the reduced velocity, the current associated with the worst-case VIM design situation does not necessarily coincide with the 100-year return period loop or hurricane current. The VIM amplitudes that induce the highest fatigue damage can occur in the presence of currents with lower return periods. The current speed profile and direction used in the single event fatigue assessment should be the most onerous speed profile and direction identified in the strength analysis. However, instead of using a constant current speed profile for the whole extreme event, current variation based on field measurements for strong loop currents can be considered. The duration of this event can be different from that obtained from the long-term current distribution.

Fatigue analysis is typically performed for the intact condition only. However, a fatigue analysis of the damaged condition should be considered for the single extreme VIM event when progressive line failure due to fatigue is a concern.

The factor of safety for fatigue design is given in 10.5. This factor should be applied to the long-term fatigue damage (due to both wind and waves and VIM) and to the single event fatigue damage under the intact condition.

8.3.5.6 VIM chain fatigue and wear

Fatigue damage of chain at the fairlead is typically higher than that away from the fairlead. For mooring systems where chain fatigue is critical, it is important to shift periodically the links at the fairlead so that additional fatigue damage due to bending can be more evenly distributed. If this procedure is part of the field operation plans, fatigue damage for the links around the fairlead can be evaluated based on the fraction of time when the links are located at the fairlead. However, the links at the fairlead should have sufficient fatigue strength to survive at least a single extreme VIM event (for example, the 100-year VIM event).

Mooring systems subjected to VIM can also experience increased wear in the links at the fairlead, which is caused by high contact pressure and large movement between links. This issue should be carefully evaluated, and the measure of periodically shifting the links at the fairlead should be considered to alleviate the wear problem. Wear measurement using go-no-go gauge as outlined in API 2I can also be considered for detecting excessive chain link wear.

8.4 Mooring line response

8.4.1 General

The floating structure dynamic response excites the mooring system in three distinct frequency bands:

— mean response;

— low-frequency responses;

— wave-frequency responses.

The extreme values of mooring line tension shall be evaluated combining these three contributions in accordance with 8.5. Extreme values of grounded lengths shall be evaluated in the same way, where applicable.

The responses of a mooring system to mean actions can be predicted by static elastic catenary equations, including line elongations. Generally, the responses to low-frequency motions can also be predicted by the same method because of the long periods of these motions. The responses of the mooring system to wave-frequency structure motions can be predicted by the methods given in 8.4.2 and 8.4.3.

8.4.2 Quasi-static analysis

In this approach, the wave actions are taken into account by statically offsetting the structure by wave-induced motions. Dynamic actions on the mooring lines associated with mass, damping and fluid acceleration are neglected. Research in mooring line dynamics has shown that the reliability of mooring designs based on this method can vary widely depending on the structure type, water depth and line configuration.

8.4.3 Dynamic analysis

Dynamic analysis of the mooring lines accounts for the time-varying effects due to mass, damping and fluid acceleration. In this approach, the time-varying fairlead motions are calculated from the structure's surge, sway, heave, roll, pitch and yaw motions. Dynamic models are used to predict mooring line responses to the fairlead motions.

Either the frequency-domain approach, 8.3.1.2, or the time-domain approach, 8.3.1.3, can be used to predict dynamic mooring tensions. In the time-domain approach, all non-linear effects including line elongation, line geometry, fluid loading, and sea floor effects can be modelled. The frequency-domain approach, on the other hand, is generally linear. Methods for approximating non-linearities in the frequency domain and their limitations should be investigated to ensure acceptable solutions for the intended application.

8.5 Line tension

8.5.1 Mean tension

The mean tension is the line tension corresponding to the mean offset of the structure response.

8.5.2 Extreme values of tension

When the frequency-domain approach is used to simulate structure response, 8.3.1.2, the extreme values of the line tension are obtained from the wave-frequency line tensions computed with the structure at positions equivalent to $S_{max} - S_{wfmax}$, with respect to each anchor location. The extreme values of line tension are then defined as

$$T_{extreme} = T_{static} \pm T_{wfmax} \tag{10}$$

where

$T_{extreme}$ is the extreme value of tension, in newtons, (N);

T_{static} is the static tension at $S_{extreme} - S_{wfmax}$, in newtons, (N);

T_{wfmax} is the maximum tension from frequency-domain analysis, in newtons, (N);

$S_{extreme}$ is the applicable value of S_{max} or S_{min} for the individual upper-terminal-point-to-anchor offset, from Equation (2) or Equation (3), respectively, in metres, (m);

S_{wfmax} is the upper-terminal-point-to-anchor offset defined in 8.2.3, in metres (m).

Alternatives to this approach are the time-domain approach, 8.3.1.3, and the combined time and frequency-domain approach, 8.3.1.4, which involve statistical processing of simulated time histories to yield maximum values of line tensions.

8.5.3 Design checks

Extreme line tension values shall be checked against the design criteria given in 10.2.

8.5.4 Tension for fatigue analysis

Line tension ranges for fatigue analysis are computed in analogy with 8.5.1 and 8.5.2. Details of the method are given in 9.3.3.2.

8.6 Line length and geometry constraints

Depending on the type of mooring system deployed, the type of anchors used and the mooring line material, a number of line length and geometry parameters shall be evaluated and assessed for compliance with design criteria.

For catenary moorings with drag anchors not specifically designed to withstand uplift, the minimum length of grounded line (always resting on the sea floor) shall be computed, and compared with a design criterion to be prescribed on a site-specific basis.

For anchors designed to withstand uplift forces, compliance with the appropriate design criteria for the specific anchor type shall be demonstrated.

For some types of mooring lines, prolonged resting on the sea floor is highly undesirable and the portion of the line closest to the anchor is typically replaced by chain. In such cases, the minimum elevation of the line-to-chain connection shall be computed, and verified against a minimum elevation criterion to be prescribed on a site-specific basis.

For some types of mooring lines, exposure to the splash zone or to friction against the fairleads is also undesirable and the upper portion of the line is typically replaced by chain. In such cases, the position of the upper termination shall be evaluated and compared with a minimum depth criterion to be prescribed on a site-specific basis.

For mooring lines in proximity to other underwater and surface installations, other clearance requirements and geometric constraints can exist. In such cases, the analysis results for the displacements at the particular points of concern shall be verified against applicable design criteria, see 10.7, or be prescribed on a site-specific basis.

Line length and geometry constraints for synthetic fibre rope lines are presented in Clause 14.

8.7 Anchor forces

The larger of the extreme tensions from the mooring analysis specified in 8.5.2 shall be used to predict maximum anchor forces. The results shall be compared with the design criteria given in 10.4, as applicable.

8.8 Typical mooring configuration analysis and assessment

8.8.1 Frequency-domain analysis for spread mooring systems

In a mooring analysis using a frequency-domain description of the structure response, the mean position of the structure is first determined from static equilibrium calculations in surge and sway directions and yaw rotation. The surge, sway and yaw responses to wave and low-frequency excitations are then calculated and added to the mean position. The procedure outlined in A.8.8.1 should be used.

Where the floating structure has a small water plane area (see 8.2.2) alternative procedures incorporating all six degrees-of-freedom should be used.

8.8.2 Frequency-domain analysis for single point mooring systems

To perform an analysis based on a frequency-domain description of the structure response, assumptions on the structure's heading shall be made. The design headings at which the mooring system responses are calculated should be determined taking into consideration the mean equilibrium heading and low-frequency yaw motions. The procedure outlined in A.8.8.2, which is likely to yield a conservative approximation, should be used.

8.8.3 Time-domain analysis

Time-domain methods may be used to perform coupled simulations of mean response, low-frequency response and wave-frequency response for the combined system consisting of the structure and moorings. This approach requires a time-domain mooring analysis tool, which solves the general equations of motion to yield the combined mean, low and wave-frequency responses of the structure, mooring lines and risers. Significant advantages of this approach are that low-frequency damping from the structure, mooring lines and risers are internally generated in the simulation, and that coupling between the structure and the mooring/riser system is fully accounted for. The procedure is presented in A.8.8.3.

8.9 Thruster-assisted moorings

8.9.1 General

The following subclauses deal with the particular configuration where a single point mooring system or a spread mooring system is assisted by onboard thrusters.

8.9.2 Analysis conditions

Intact and redundancy check definitions for thruster-assisted moorings (TAM) are given in Table 2 below.

Table 2 — Intact and redundancy check TAM definitions

TAM definition	Mooring system condition	Thruster system condition
Intact	Intact	Intact
Redundancy check	Intact	Redundancy check
Redundancy check	Redundancy check	Intact

8.9.3 Determination of allowable thrust

When thrusters are used to assist the mooring system, the allowable thrust to be used in the mooring analysis shall be determined as follows.

a) Determine the available effective thrust, taking into consideration the efficiency of the thrusters and losses due to floating structure motions, current, thruster/hull and thruster/thruster interference effects, and any directional restrictions.

b) Determine the worst thruster system failure. FMEA should be performed to identify the worst single failure, see 13.2.1. The definition of the worst single failure should allow for thruster system availability (mean time to failure and mean time to repair) over the design service life of the installation.

c) Determine the allowable thrust:

   1) for automatic thruster control systems, the allowable thrust shall be either

      i) for the intact thruster condition, equal to the available effective thrust, or

      ii) for the redundancy check thruster condition, equal to the available effective thrust after accounting for the worst failure as determined by the FMEA;

   2) for manual thruster control systems, the allowable thrust shall be 0,7 of the value found in 1).

The allowable thrust used in the mooring analysis should be verified during thruster system sea trials.

8.9.4 Load sharing

8.9.4.1 General

In a TAM system, load sharing between the thruster and the mooring systems is complex. However, the simple mean load reduction method described below yields reasonable results.

8.9.4.2 Mean load reduction method

In this simplified approach, the thrusters are assumed to counter only the mean environmental actions in the surge, sway and yaw directions. Allowable thrusts from thrusters are first evaluated, see 8.9.3, and then subtracted from the mean action. The remainder of the mean action and the wave and low-frequency motions shall be sustained by the mooring system.

For structures with spread moorings, where the floating structure heading is held stable by the mooring lines, the surge and sway components of the allowable thrust can be subtracted from the mean surge and mean sway environmental actions. The mooring response shall then be evaluated using the analysis procedure for mooring systems without thruster assistance, see 8.8.1 to 8.8.3.

8.9.4.3 Heading control and surge damping

For structures with single point moorings, the main function of the thrusters is heading control. For structures with high thruster capacity that significantly exceeds the heading control requirements, the available capacity may be shared between heading control and the generation of low-frequency surge damping.

8.9.4.4 System dynamic analysis

A system dynamic analysis should be performed in the time domain. This model should generate the mean offset, low-frequency structure motions, control system characteristics and thruster responses corresponding to specific environmental conditions.

8.10 Transient analysis of floating structure motions

8.10.1 General

A moored floating structure experiences transient motions after a mooring line breakage or thruster system failure before it settles about a new equilibrium position, see 8.1.2.4. Transient analysis shall be performed to check the maximum offset resulting from such events, in accordance with Table 1.

Transient analysis of a moored structure under wind, wave, current and thruster actions is complex and can require a time-domain solution. To simplify the analysis, a combined approach may be used as described in 8.10.2.

8.10.2 Combined time and frequency-domain analyses

In this approach, the maximum transient motion is first determined using a time-domain analysis. The structure low and wave-frequency motions obtained from a frequency-domain analysis are then combined with the transient motions. The recommended procedure is as follows.

a) Compute the equilibrium position under mean actions for the intact mooring.

b) Simulate a line breakage and compute the maximum transient motion (overshoot) in the time domain with mean load only, but with the mooring system stiffness updated at each time step. Generally, a model with three degrees-of-freedom (surge, sway and yaw) is required.

c) Determine maximum structure offset by

$$S_{\text{max}} = S_{\text{mean}} + S_{\text{lfsig}} + S_{\text{wfsig}} + S_{\text{t}} \tag{11}$$

where

S_{mean} is the mean offset as calculated in step a), in metres (m);

S_{lfsig} is the positive value of significant low-frequency motion, calculated in the frequency domain using the redundancy check mooring system stiffness, in metres (m);

S_{wfsig} is the positive value of significant wave-frequency motion, calculated in the frequency domain, in metres (m);

S_t is the maximum transient motion (overshoot) with respect to the equilibrium position from step a), as determined in step b), in metres (m).

8.10.3 Time-domain analysis

Time-domain transient analysis is similar to the structure response time-domain analysis described in 8.3.1.3. The only difference is that a mooring line is removed during the simulation to model a line breakage. The simulation should be repeated for a number of wave action time histories and for the break to occur at different time steps for each time history. The maximum offset observed during these simulations, or the maximum offset estimated from the results of these simulations, see 8.3.2, should be used.

9 Fatigue analysis

9.1 Basic considerations

Fatigue assessment is a complex process with many uncertainties. Typical fatigue analysis procedures are presented herein. Alternative procedures may be used, provided they can be documented to achieve similar levels of reliability to those presented herein.

Miner's Rule shall be used to determine accumulated fatigue damage. For the main components of mooring lines (i.e. chain, wire rope and connecting links), Miner's Rule calculations may be based on tension range (the so-called T-N approach) as described below, or on the stress range (the so-called S-N approach) as described in A.9.1. For other components, such as anchor piles and attachments, the S-N approach is generally used.

Time- and/or frequency-domain dynamic analyses shall be used to determine tension or stress ranges. A quasi-static analysis may be used only if it can be fully documented to achieve levels of reliability similar to those obtained with the time and/or frequency-domain dynamic analyses. Tank model test data may be used in lieu of dynamic analyses provided these data are fully documented as being suitable for fatigue analysis.

The determination of the fatigue resistance of some typical mooring system components is specified in 9.2. The fatigue resistance of synthetic fibre rope is discussed in Clause 14.

Calculation procedures for the determination of tension range cycles are given in 9.3. Several approaches for determining fatigue damage, involving varying degrees of simplification in the damage summation by the Miner's Rule, are given.

9.2 Fatigue resistance

9.2.1 Wire rope, chain and connecting links

The fatigue resistance of wire rope, chain, connecting links and other mooring system components is given by T-N curves, where the tension range, T, is usually non-dimensionalized by dividing by a suitable reference breaking strength and N is the permissible number of cycles. T-N curves shall be based on fatigue test data for these components and a regression analysis.

Representative T-N curves for some wire rope, chain and connecting links are given in 9.2.2. T-N curves shall be used to assess tension-tension (T-T) fatigue conditions according to 9.2.3. Bending-tension (B-T) and free bending fatigue conditions are addressed in 9.2.4.

9.2.2 T-N curves

The equation for the representative T-N curve is

$$N\,T^{m} = K \tag{12}$$

where

N is the number of permissible cycles of tension range ratio, T;

T is the ratio of tension range (double amplitude) to the reference breaking strength of the component (the appropriate reference strength for each type of component is given below);

m is the inverse slope of the T-N fatigue curve;

K is a constant.

T-N curves plot as straight lines on log-log paper.

Values for m and K are given in Table 3 for a limited selection of chain link, connecting link and wire rope components.

Table 3 — Values of m and K for representative T-N curves

Component	m	K
Common studlink	3,00	1 000
Common studless link	3,00	316
Baldt and Kenter connecting link	3,00	178
Six / multi-strand wire rope (corrosion protected)	4,09	$10^{(3,20 - 2,79\,Q)}$ = 231, when Q = 0,3
Spiral strand wire rope (corrosion protected)	5,05	$10^{(3,25 - 3,43\,Q)}$ = 166, when Q = 0,3
NOTE Q is the ratio of mean tension to MBS of the wire rope.		

The reference breaking strength for R3, R4, and R4S common or connecting links is equal to the minimum breaking strength (MBS) of ORQ (Oil Rig Quality) common chain link of the same diameter.

To allow for the effects of corrosion and wear when determining the reference breaking strength of R3, R4, and R4S common or connecting links, the diameter should be taken equal to the nominal diameter minus half of the corrosion and wear allowance.

Table 3 indicates that the fatigue resistance of studless chain is less than that of the corresponding studlink chain. However, the presence of studs introduces a number of possible fatigue issues that cannot be detected by inspection (i.e. loose stud, stud weld crack, sharp corners at stud footprint, corrosion between stud and link, and defects hidden behind the stud). The equation for studlink chain is not valid for links with loose studs. Consequently, it is important to consider all factors affecting fatigue resistance in the selection of chain type.

The T-N curve for Baldt and Kenter connecting links is based on limited fatigue test data.

On the basis of limited fatigue data, the fatigue resistance of D-shackles is comparable to that of common links of the same size and grade, provided the shackle is machined to fit with close tolerance, no cotter pin is used through the shackle body, and the shackle is of the narrow throat type.

For other types of connecting links, where published data are insufficient to generate fatigue resistance curves, appropriate tests shall be performed to establish such curves prior to the use of such links in permanent mooring systems. The tests shall be performed in a manner consistent with the tests on which the T-N curves listed in Table 3 are based.

The reference breaking strength for wire rope is equal to its MBS.

The T-N curves for wire rope are based on fatigue test data for six-strand, multi-strand and spiral strand rope. As shown in Table 3, wire rope fatigue resistance is a function of the mean tension in the rope which, in a catenary mooring system, is typically 0.2 to 0,3 of MBS. The fatigue assessment of wire rope shall account for the effect of mean tension according to 9.3.3.

The wire rope fatigue data are based on tests in which the effects of corrosion have been excluded. The T-N curves in Table 3 are therefore only appropriate for wire ropes protected from corrosion by, for example, galvanizing, sheathing, blocking compound and zinc filler wires. When using wire rope as part of permanent mooring systems, the design service life, inspection, and change out strategy should all be considered when determining the combination of corrosion protection systems needed for a specific application.

9.2.3 Tension-tension (T-T) fatigue

Chain, connecting link and wire rope components of mooring systems subjected to tension only shall be designed on the basis of fatigue resistance determined in accordance with 9.2.2 for the fatigue damage calculated in accordance with 9.3.3. The fatigue assessment of other mooring system components subjected to pure tension that can be designed using T-N curves should also be conducted in accordance with 9.2.2 and 9.3.3, provided appropriate T-N curves exist or can be developed.

9.2.4 Bending-tension (B-T) and free bending fatigue

Combined bending-tension of wire rope and chains generally occurs at locations such as fairleads, bending shoes, chain stoppers, hawser pipes, bend-limiting devices, and adjacent to clump weights and mid-water buoys. At these locations, tension-tension fatigue damage is aggravated by the presence of bending as well as by the additional possible effects of wear and corrosion.

In the absence of suitable data on the fatigue damage due to bending-tension on wire ropes, the bend diameter to rope diameter ratio should be large enough to avoid excessive bending, see Table A.4.

Fatigue analysis of wire rope and chains shall adequately account for the additional stress concentration at points of direct contact. Fatigue damage under such conditions can be alleviated by a combination of regular inspection and adjustment of the line so as to avoid concentrated bending at one location.

Free bending at wire rope terminations can induce significant fatigue damage and reduce fatigue life. Bend-limiting devices should be incorporated at such locations. Such devices shall be designed to smoothly transfer forces from the termination to the rope over the full range of structure draft and offset conditions.

9.3 Fatigue analysis procedure

9.3.1 General

The line tension calculation procedure for fatigue analysis is given in 9.3.3.2. Recommended fatigue analysis procedures are described in 9.3.2 and 9.3.3, while the fatigue safety factor is given in 10.5.

9.3.2 Cumulative fatigue damage

The total fatigue damage during the design service life, L, is assumed to be equal to L times the annual fatigue damage. The annual fatigue damage in a mooring line component shall be determined as the sum of the annual fatigue damage arising in a combination of n mooring design states (MDSs). Each MDS consists of

— a directional sea state chosen to discretize the long-term environment to which the mooring system is subjected,

— a probability of occurrence of the sea state,

— a mean offset and heading representing the effects of the sea state, with associated wind and current, and

— representative loading conditions (i.e. draft conditions) of the floating structure.

The annual fatigue damage shall be computed as

$$D = \sum_{i=1}^{n} D_i \tag{13}$$

where

D is the annual fatigue damage in a mooring line component, years^{-1};

D_i is the annual fatigue damage arising in MDS$_i$, years^{-1}.

The discretization into i = 1, …, n combinations should take into account the sensitivity of the fatigue calculation to the assumed input parameters, including the effects of

— variation in direction of resultant mean environmental action,

— significant wave height,

— spectral peak period or mean zero-crossing period,

— spectral peakedness,

— mean floating structure offset (due to slow drift, current or other effects),

— hydrodynamic damping due to current or slow drift motion,

— expected range of current profiles with depth, and

— floating structure loading conditions.

9.3.3 Fatigue damage assessment

9.3.3.1 General

Once an appreciation of the overall configuration and the influence of environmental input data noted in 9.3.2 have been established, representative data for each MDS shall be selected and documented as the basis of the long-term fatigue assessment. Each environmental condition shall be defined in terms of the wind, wave and current parameters and their directions. Floating structure loading conditions shall be identified in terms of draft, trim and heel (corresponding to a particular distribution of storage and ballast).

The annual probability of occurrence, P_i, of MDS$_i$ shall be defined. Typically, eight to twelve directions should be considered as representing the directional distribution of the long-term environment. A sensitivity assessment, as described in 9.3.2, can be used to identify the number of sea states needed for an adequate long-term representation — typically 10 to 50. Normally, three floating structure loading conditions should be sufficient: floating structure fully loaded, fully ballasted and a representative long-term "mean" operating condition. If the mooring system operates in different modes, for example, with and without a structure attached, a separate analysis for each mode shall be performed.

The annual fatigue damage accumulated in an individual MDS shall be computed as

$$D_i = \left[\sum_k \frac{n_k}{N_k} \right]_i \tag{14}$$

where

n_k is the number of cycles of the non-dimensional tension range T_k occurring in MDS$_i$, per year;

N_k is the number of permissible cycles for non-dimensional tension range T_k from Equation (12), per year;

T_k is the non-dimensional tension range, as defined in 9.2.2.

The number of tension range cycles per year in each MDS can be determined as

$$n_i = f_i X_i = f_i \cdot P_i \cdot C_1 \tag{15}$$

where

f_i is the zero up-crossing frequency of the tension spectrum in MDS$_i$, in hertz;

X_i is the time spent in MDS$_i$ per annum, s;

P_i is the probability of occurrence of MDS$_i$;

C_1 is the average number of seconds per annum = 3,15576 × 10^7 s.

NOTE A simplified formulation of Equation (14) is presented in A.9.3.3.1, where T_k follows a Rayleigh distribution and the T-N curve is consistent with the form of Equation (12).

9.3.3.2 Tension range calculation

When performing frequency-domain analysis, the standard deviation of low-frequency tension shall be calculated about the mean floating structure offset for each MDS, i.e.

$$s_{\text{lf\_eff}} = T_{\text{lf\_mean + stdev}} - T_{\text{mean}} \tag{16}$$

where

$s_{\text{lf\_eff}}$ is the effective standard deviation of the low-frequency tension, in newtons (N);

$T_{\text{lf\_mean + stdev}}$ is the tension at $S_{\text{mean}} + s_{\text{lf\_s}}$, in newtons (N);

T_{mean} is the tension at the mean offset, in newtons (N);

S_{mean} is the mean structure offset, in metres (m), as defined in 8.2.2;

$s_{\text{lf\_s}}$ is the standard deviation of low-frequency motion of the upper-terminal-point-to-anchor offset, for each line, in metres (m).

The standard deviation of wave-frequency tension shall also be calculated about the mean floating structure offset for each MDS.

When performing time-domain analysis, the time history of the tension can be directly derived from the analysis.

9.3.3.3 Combining wave-frequency and low-frequency tensions

9.3.3.3.1 General

For fatigue damage assessment, the following four methods can be considered for combining the damage due to wave-frequency and low-frequency tensions.

a) Simple summation: wave-frequency damage and low-frequency damage are calculated independently and the total damage is taken as the sum of the two.

b) Combined spectrum: the standard deviations of wave-frequency tension ranges and low-frequency tension ranges are calculated independently based on the separate wave-frequency and low-frequency tension spectra or the separate wave-frequency and low-frequency tension time series; the standard deviation of the combined response is computed using Equation (19). The damage is then calculated using the combined standard deviation.

c) Combined spectrum with dual narrow-banded correction factor: a correction factor is applied to the result of the combined spectrum method presented in b).

d) Time-domain cycle counting: fatigue damage is calculated from a tension time history using a cycle counting method, such as the rainflow method, to estimate the magnitude and number of tension range cycles. The tension time history can be determined directly by a time-domain analysis or it can be generated from the combined low and wave-frequency spectral analyses.

The relative merits of each of these methods are as follows.

— Method a), simple summation, is in principle non-conservative, but may be applied to a MDS if the low-frequency tension contribution to the total tension response is negligible [$\lambda_{\mathrm{L}i} < 0{,}15$, see Equation (21)].

— Method b), combined spectrum, is generally conservative and may be applied to any MDS.

— Method c), combined spectrum with dual narrow-banded correction factor, is an improvement that yields less conservative predictions than method b), and that may also be applied to any MDS. However, the improvement tends to be lost in a MDS where the low-frequency tension is strongly dominant.

— Method d), time-domain cycle counting, if rigorously performed with sufficient number of time simulations representative of the wave scatter diagram, is generally considered to be the most accurate method of calculating fatigue damage and should be used when time history of the combined low-frequency and wave-frequency tension is available.

Analysis procedures for methods a), b) and c) are presented in 9.3.3.3.2 to 9.3.3.3.4.

9.3.3.3.2 Simple summation

The annual fatigue damage D_i for state MDS$_i$ shall be determined by separate application of Equations (14) and (15) to wave-frequency and low-frequency tension ranges. The result is given in Equation (17):

$$D_i = \frac{n_{Wi}}{K}\left(2\sqrt{2}\sigma_{Wi}\right)^m \times \Gamma\left(1+\frac{m}{2}\right) + \frac{n_{Li}}{K}\left(2\sqrt{2}\sigma_{Li}\right)^m \times \Gamma\left(1+\frac{m}{2}\right) \tag{17}$$

where the symbols are as defined in 9.2.2. Symbols with subscript Wi relate to the tensions for MDS$_i$ relative to wave-frequency excitation only, while symbols with subscript Li relate to the tensions for MDS$_i$ due to low-frequency excitation only, and where

n_{Li} is the number of low-frequency tension range cycles per year for MDS$_i$ from Equation (15) in which the average zero up-crossing frequency can be estimated by $1/t_\mathrm{n}$, where t_n is the natural period (surge, sway and/or yaw as appropriate) of the moored structure computed at its mean position;

n_{Wi} is the number of wave-frequency tension cycles per year for MDS$_i$ from Equation (15);

σ_{Li} is the ratio of the standard deviation of low-frequency tension variations around the mean tension to the reference breaking strength;

σ_{Wi} is the ratio of the standard deviation of wave-frequency tension variations around the mean tension to the reference breaking strength;

Γ is the Gamma function.

K and m are defined in 9.2.2.

NOTE The standard deviation of the tension range probability distribution is twice the standard deviation of tension.

9.3.3.3.3 Combined spectrum

The annual fatigue damage D_i for state MDS$_i$ shall be determined as follows, based on a Rayleigh distribution of tension peaks:

$$D_i = \frac{n_i}{K}\left(2\sqrt{2}\sigma_i\right)^m \times \Gamma\left(1+\frac{m}{2}\right) \tag{18}$$

where

σ_i the ratio of the standard deviation of the combined wave and low-frequency tension variations around the mean tension to the reference breaking strength

$$\sigma_i = \sqrt{\sigma_{Wi}^2 + \sigma_{Li}^2} \tag{19}$$

n_i is the number of tension range cycles per year for MDS$_i$ from Equation (15) in which the average zero up-crossing frequency of the combined spectrum is given by

$$f_{Ci} = \sqrt{\lambda_{Wi} f_{Wi}^2 + \lambda_{Li} f_{Li}^2} \tag{20}$$

f_{Wi} is the zero up-crossing frequency of the wave-frequency tension spectrum in MDS$_i$, in hertz (Hz);

f_{Li} is the zero up-crossing frequency of the low-frequency tension spectrum in MDS$_i$, (calculated as described under n_{Li} in 9.3.3.3.2), in hertz (Hz);

$$\lambda_{Li} = \frac{\sigma_{Li}^2}{\sigma_{Li}^2 + \sigma_{Wi}^2} \tag{21}$$

$$\lambda_{Wi} = \frac{\sigma_{Wi}^2}{\sigma_{Li}^2 + \sigma_{Wi}^2} \tag{22}$$

9.3.3.3.4 Combined spectrum with dual narrow-banded correction factor

The annual fatigue damage for state MDS$_i$ shall be determined from

$$D_i = \rho_i \frac{n_i}{K}\left(2\sqrt{2}\sigma_i\right)^m \times \Gamma\left(1+\frac{m}{2}\right) \tag{23}$$

where

$$\rho_i = \frac{f_{ei}}{f_{Ci}}\left[\left(\lambda_{Li}\right)^{\left(2+\frac{m}{2}\right)}\left(1-\sqrt{\frac{\lambda_{Wi}}{\lambda_{Li}}}\right)+\sqrt{\pi\lambda_{Li}\lambda_{Wi}}\,\frac{m\Gamma\left(\frac{1+m}{2}\right)}{\Gamma\left(1+\frac{m}{2}\right)}\right]+\frac{f_{Wi}}{f_{Ci}}\left(\lambda_{Wi}\right)^{\frac{m}{2}} \tag{24}$$

f_{ei} is the mean up-crossing frequency of the envelope of the normalized tension variations around the mean tension

$$f_{ei} = \sqrt{\lambda_{Li}^2 f_{Li}^2 + \lambda_{Li}\lambda_{Wi} f_{Wi}^2 \delta_{Wi}^2} \tag{25}$$

δ_{Wi} is the bandwidth parameter for the wave-frequency component of the normalized tension variations around the mean tension, which may be taken as equal to 0,1.

Values of the gamma function for some typical values of m are given in Table 4.

Table 4 — Gamma functions

m	**3,00**	**4,09**	**5,05**
$\Gamma\left[1+\frac{m}{2}\right]$	1,329	2,086	3,417
$\Gamma\left[\frac{1+m}{2}\right]$	1,000	1,373	2,047

9.3.3.4 Accounting for mean value of tension in wire rope

Fatigue damage in wire rope depends on the mean value of the tension as well as on the time-varying component. To account for this dual effect the mean tension shall first be determined for each mooring line in each MDS. From Table 3, the corresponding representative T-N curve shall then be selected to calculate the fatigue damage for that MDS. This implies that in principle a different representative T-N curve is applicable for each mooring line in each MDS.

10 Design criteria

10.1 Floating structure offset

Floating structure offset limits shall be established by clearance requirements and limitations on the satisfactory performance of equipment such as umbilicals, risers and gangways, and the time required for the safe operation of any disconnect system. Generally, different criteria apply to intact, redundancy check and transient motion conditions and these are detailed below.

The offset of the floating structure from the sea floor well location shall be controlled to prevent damage to drilling, well intervention or production risers.

10.2 Line tension limit

For a mooring component, a tension limit should be expressed as a percentage of its MBS after reductions for corrosion and wear.

Tension limits for various conditions and analysis methods shall be set in accordance with Table 5, in which design safety factors are also listed. These limits apply only to properly maintained moorings and systems in which the connecting components have MBS greater than or equal to that of the mooring lines.

The same mooring line tension limits apply to TAM, provided that the thruster system is designed to an appropriate level of reliability and is capable of making a significant contribution to the stationkeeping capacity of the floating structure.

Table 5 — ULS line tension limits and design safety factors

Analysis condition	Analysis method	Line tension limit (percent of MBS)	Design safety factor
Intact	Quasi-static	50 %	2,00
Intact	Dynamic	60 %	1,67
Redundancy check	Quasi-static	70 %	1,43
Redundancy check	Dynamic	80 %	1,25
Transient	Quasi-static or dynamic	95 %	1,05

10.3 Grounded line length

If drag anchors are used, the outboard mooring line length shall be sufficient to prevent anchor uplift under any of the conditions covered by 8.1.2.5, unless it can be demonstrated that the anchor has sufficient vertical resistance. Guidance on the use of drag anchors to resist vertical forces is provided in A.10.4.2.

Shorter grounded line lengths may be used for moorings with anchoring systems that can resist substantial vertical uplift such as anchor piles, suction caissons, or anchors with demonstrably adequate vertical resistance.

10.4 Anchoring systems

10.4.1 General

The options that are available for anchoring floating structures include

— drag anchors,

— anchor piles (driven, jetted, suction, torpedo (gravity-embedded) and drilled and grouted), and

— other anchor types such as gravity anchors and plate anchors.

In selecting anchor options, consideration should be given to required system performance, soil conditions, reliability, installation and the test load. The structural strength of anchors should be demonstrated to be adequate with respect to the required foundation capacities.

10.4.2 Drag anchors

The ultimate holding capacity of a drag anchor represents the maximum horizontal component of the steady pull that can be resisted by the anchor at continuous drag. This includes the soil resistance on the buried portion of the chain or of the wire rope, but excludes the friction of the chain or wire rope lying on the sea floor.

For typical mooring applications and soil conditions (i.e., homogeneous soft to medium stiff clay), the larger of the extreme mooring line tensions shall not exceed the ultimate holding capacity divided by the design safety factor given in Table 6.

Table 6 —ULS drag anchor holding capacity design safety factors

		Quasi-static analysis	Dynamic analysis
Permanent mooring	Intact condition	N/A	1,50
	Redundancy check	N/A	1,00
Mobile mooring	Intact condition	1,00	0,80
	Redundancy check	Not required	Not required

The factors in Table 6 do not necessarily apply to drag embedded plate anchors, which are further discussed in A.10.4.4.2.1.4.

10.4.3 Anchor piles

The representative vertical and horizontal resistances of anchor piles should be determined in accordance with the requirements for fixed steel structures, as applicable. The vertical and horizontal components of the larger of the extreme mooring line tensions derived from dynamic analysis shall not exceed the representative resistances divided by the design safety factors given in Table 7.

Table 7 — ULS design safety factors for holding capacity of anchor piles and suction piles

	Permanent mooring		Mobile mooring	
Analysis condition	**Axial loading**	**Lateral loading**	**Axial loading**	**Lateral loading**
Intact condition	2,00	1,60	1,50	1,20
Redundancy check	1,50	1,20	1,20	1,00

10.4.4 Other anchor types

Design criteria for gravity anchors and plate anchors are given in Table 8. Other anchor types may be deployed provided they can be documented to achieve similar levels of reliability to the anchors discussed in Tables 6, 7 and 8. Design safety factors are defined as anchor capacity from the failure envelope divided by extreme values of the anchor forces from dynamic analysis.

Table 8 — ULS design safety factors for holding capacity of gravity and plate anchors

	Gravity anchor				Plate anchor	
	Permanent mooring		Mobile mooring		Permanent mooring	Mobile mooring
Analysis condition	**Axial loading**	**Lateral loading**	**Axial loading**	**Lateral loading**		
Intact condition	2,00	1,60	1,50	1,20	2,00	1,50
Redundancy check	1,50	1,20	1,20	1,00	1,50	1,20

10.4.5 Chain and wire rope holding capacity

The holding capacity from friction of chain and wire rope on the sea floor may be estimated using:

$$F_{cw} = C_f \, l_{cw} \, W_{cw} \tag{26}$$

where

F_{cw} is the chain or wire rope holding capacity, in newtons (N);

C_f is the coefficient of friction between chain or wire rope and the sea floor;

l_{cw} is the length of chain or wire rope in contact with the sea floor, in metres (m);

W_{cw} is the submerged unit weight of chain or wire rope, in newtons per metre (N/m).

10.4.6 Mooring test load

10.4.6.1 General

All moorings shall be subject to load testing prior to initial use and again after any substantive change to the mooring configuration, whether by intent or by some natural or accidental event. The purpose of load testing is to ensure adequate holding capacity of the anchoring system, eliminate slack in the grounded portion of the mooring lines, and allow detection of any significant installation-induced damage to mooring components. The requirements for such test loads shall be defined by the designer in accordance with the following subclauses. Further details on test loading of suction and plate anchors are provided in A.10.4.6.1.

Note that the mooring test load should also be set high enough such that the additional drag to resist the design intact actions does not overload neighbouring lines. This is further discussed in A.10.4.2.1, A.10.4.2.10 and A.10.4.2.11.

10.4.6.2 Permanent moorings

For mooring lines with drag anchors, the test load magnitude in soft clay where deep anchor penetration can be achieved shall be equal to at least 80 % of the force induced by the environmental design situation as determined by a dynamic analysis of the intact condition (see A.10.4.6.2). In hard, sandy, or layered soils, where anchor penetration can be limited to no more than one fluke length, the test load magnitude shall be higher, and should be 100 % or more of the force induced by the environmental design situation as determined by a dynamic analysis of the intact condition. In defining the test load, the designer should consider the uncertainties in the calculated force and the consequences of potential platform displacement resulting from anchor movement.

For mooring lines with fixed anchors, e.g. anchor piles, the test load magnitude shall be sufficient to ensure the correct setting of the mooring line assembly and to ensure that the inverse catenary is sufficiently formed to prevent unacceptable mooring line slacking due to additional cut-in of the inverse catenary during storm conditions.

The duration of the installation test load shall be at least 15 min.

Note that that installation tension should also be set high enough such that the additional drag to resist the design intact actions does not overload neighbouring lines. This is further discussed in A.10.4.2.1, A.10.4.2.10 and A.10.4.2.11.

10.4.6.3 Mobile moorings

For mooring lines with drag anchors, the installation test load magnitude should be determined by consideration of a number of factors, including type of anchor, soil conditions, winch pull limit, and anchor retrieval. As a minimum, the following shall be satisfied:

— the installation test load at a winch shall not be less than the larger of the extreme line tensions for an intact mooring under the design situation for SLS;

— the installation test load at the anchor shank shall not be less than three times the anchor weight;

— for moorings in proximity to other installations, the installation test load at a winch shall not be less than the mean line tension for an intact mooring under the design situation for ULS;

— the duration of the installation test load shall be at least 15 min.

10.5 Fatigue safety factor

The total fatigue damage shall satisfy

$$D_{\mathrm{T}}\ \gamma_{\mathrm{F}} \le 1{,}0 \qquad (27)$$

where

D_{T} is the total accumulated damage from all sources over the life cycle of the stationkeeping system;

D_{T} $= D\ L$ + any fatigue damage arising from other sources;

D is the annual fatigue damage calculated in accordance with Clause 9, years^{-1};

L is the design service life (from 9.3.2), in years;

γ_{F} is the fatigue safety factor.

The minimum value of γ_{F} shall be 3,0.

10.6 Corrosion and wear

Protection against corrosion and wear (including fretting) shall be provided for permanent mooring systems.

For chain, a corrosion and wear allowance is provided by an appropriate increase in the link diameter. The increase shall be determined by a site-specific assessment dependent upon several parameters, e.g. water salinity. Typical values of corrosion and wear allowance are

— 0,2 mm to 0,8 mm per year of the design service life, for those parts of a mooring line in the splash zone or zone of hard-bottom sea floor contact, and

— 0,1 mm to 0,2 mm per year of the design service life, for the remaining length.

Corrosion of wire rope at connections to sockets can be accelerated by the galvanized wire acting as an anode for adjacent components. For permanent systems, either the wire shall be electrically isolated from the socket, or the socket shall be isolated from the adjacent component. Additional corrosion protection can be achieved by the addition of sacrificial anodes to this area.

10.7 Clearances

10.7.1 Basic considerations

Clearances between a floating structure or its mooring components and other installations shall be in accordance with national and/or state regulations, as applicable. Where no other guidance exists, clearances determined for the conditions specified in 8.1.2.5 shall satisfy the requirements given in 10.7.2 to 10.7.4.

10.7.2 Mooring line crossing pipeline

Where a mooring line within the elevated part of its catenary crosses a pipeline on the sea floor, a minimum vertical clearance of 10 m under the intact condition shall be maintained. A mooring line may pass over and be in contact with a protected pipeline provided this contact is not interrupted throughout the full range of predicted intact line tensions, i.e. contact does not occur in the thrash zone.

10.7.3 Horizontal distance between installations

A minimum horizontal clearance of 10 m shall be maintained between a floating structure, including its mooring lines, and any other installation for all relevant conditions according to 8.1.2.5.

The clearance requirement may be reduced following an appropriate risk assessment.

10.7.4 Clearance between a drag anchor and other installations

If the drag path of a drag anchor to a floating structure is expected to bring it within close proximity to another installation, the final anchor position shall be such as to allow a margin of at least 300 m of drag before contact can occur with the installation. Otherwise, the final anchor position shall be at least 100 m from the installation.

10.8 Supporting structures

The representative resistances of supporting structures such as chain stoppers, fairleads and their foundations shall be larger than those of the mooring line components. Special attention shall be given to the design of supporting structures such that their failure will not result in multiple-line failure.

11 Mooring hardware

11.1 Mooring line components

11.1.1 General

Manufacturing of mooring hardware should be subject to an appropriate level of quality assurance.

11.1.2 Wire rope

Mooring wire rope shall not have a fibre core.

The spaces between wires to a rope shall be filled with blocking compound of good quality.

The ends of each rope section shall terminate in resin- or zinc-poured sockets.

In all other respects, mooring wire ropes and end sockets shall meet the material, design, manufacturing and testing requirements specified in relevant RCS rules, see A.11.1.2.

11.1.3 Chain

Mooring chain shall be manufactured in accordance with appropriate rules for offshore mooring chain, see A.11.1.3.

11.1.4 Connecting links

Connecting links shall be manufactured in accordance with appropriate rules for offshore mooring chain, see A.11.1.4.

11.1.5 Spring buoys

Surface spring buoys shall be designed to remain at the surface (i.e. a maximum 67 % submergence) in all intact and redundancy check conditions, unless designed for the appropriate maximum submergence. They shall incorporate measures such as compartmentalization, in order to minimize the risk of sinking in the event of damage.

Subsurface spring buoys of steel construction, designed for external pressure, shall be designed in accordance with a recognized pressure containment standard for use at the maximum operational depth identified by the mooring analysis. The design safety factor shall be no less than 1,5 for permanent moorings and 1,2 for mobile moorings. Further guidance on pressure containment standards and design safety factors are given in A.11.1.5.

For foam type buoys, an allowable hydrostatic pressure shall be determined by dividing the hydrostatic collapse pressure by a design safety factor. The choice of the design safety factor depends on the buoy's permanence and criticality.

Buoy motions shall be considered in the design of connecting links to the buoy.

11.1.6 Anchors

The anchor options referred to in 10.4.1 include

— drag anchors,

— anchor piles (driven, jetted, suction, torpedo (gravity-embedded) and drilled and grouted), and

— other anchor types such as gravity anchors and plate anchors.

Details of these anchors are described in A.11.1.6.

11.2 Winching equipment

Winches should meet the provisions specified in internationally recognized design standards, see A.11.2.

Mooring lines are subjected to high wear and high stresses at the fairlead and stopper arrangements. Fairleads and stopper arrangements should be designed to minimize wear and fatigue of mooring line components.

11.3 Monitoring equipment

11.3.1 Line tension

Moored floating structures shall be equipped with a calibrated system for measuring mooring line tensions if the operation requires mooring line adjustment, and line tensions shall be continuously displayed at each winch. For permanent floating structures that do not require a tension measurement device, a means of detecting mooring failure shall be installed.

For structures with thrusters that are intended for mooring line tension reduction, a means of indicating line tension and/or floating structure offset shall be provided. This means should be suitably redundant to cover the single-line failure condition.

11.3.2 Line payout

If operation of the floating structure requires mooring line adjustment, mooring lines shall be equipped with a system for measuring line payout.

11.3.3 Floating structure position

If serviceability requirements impose constraints on the floating structure offset, the structure shall be equipped with a system to monitor its position. For MODUs, a positioning system shall be deployed to monitor the structure's distance from the wellhead or the point of riser attachment.

For floating structures with TAM, position measurement shall be suitably redundant to cover the single failure conditions.

11.3.4 Floating structure heading

Floating structures with a single point mooring and MODUs shall be equipped with instrumentation to monitor heading.

If the heading is to be controlled, at least two different heading references shall be provided.

If heading control is automatic, the accuracy and update rate of both devices shall be adequate to meet automatic control requirements. If heading control is critical, three independent heading references shall be provided so that a drifting reference can be rejected without a heading excursion.

For MODUs, reference should be made to appropriate IMO and IMCA specifications, see A.11.3.

12 In-service inspection, monitoring and maintenance

12.1 General

Requirements for in-service inspection, monitoring and maintenance of mobile moorings, which are routinely retrieved and redeployed, are well established, see 12.2. Permanent moorings are expected to remain in-place for their entire design service life. Measures for their long-term in-service inspection, monitoring and maintenance, which shall encompass the full stationkeeping function, can be effectively addressed through the use of a structural integrity management (SIM) system in accordance with 12.3.

12.2 Mobile moorings

In-service inspection of mooring line components should comply with appropriate IMO, RCS rules or other relevant documents for MOUs, see A.12.2.

12.3 Permanent moorings

12.3.1 General

The owner shall ensure that suitable arrangements are in place for monitoring and maintaining the integrity of a stationkeeping system throughout its design service life. Such arrangements include planned maintenance and inspection of the system, periodic assessment of its condition in relation to original design expectations, assessment of damage or suspected damage, and arrangements for repair and/or change-out in the event of damage or deterioration. Periodic assessments should reflect current good practice and incorporate advances in knowledge and changes in risk level as appropriate. The frequency, scope and methods of inspection shall be sufficient to provide assurance, in conjunction with associated assessments, that the integrity of the mooring system is being maintained.

Replacing stationkeeping components can be time consuming and involve additional risks, particularly if diver intervention is required. Component replacement as part of the maintenance programme should be avoided.

The purpose of the SIM system is to provide a formal process for ensuring the integrity of the stationkeeping system throughout its intended design service life. This requires the provision of safe and effective means of inspecting, monitoring and maintaining the system and the repair and/or change-out of components, in accordance with the philosophy described in 5.4.

12.3.2 Structural integrity management system philosophies

12.3.2.1 General

Approaches for dealing with structural integrity management will vary depending upon field life, type of floating structure, mooring system configuration, and sophistication or otherwise of local infrastructure. In turn, these factors influence the philosophical approach to the specification of a SIM system. The philosophy can vary from one involving emphasis on the use of monitoring equipment to one with a preference for the extensive use of inspections. Some prefer a proactive approach to component replacement, whereas others follow a replace-when-broken philosophy. Clearly, differences in philosophy lead to differences in the manner in which the SIM system is developed and implemented. Irrespective of the philosophy, the resulting SIM system shall be conceived with the aim of maintaining the integrity of the mooring system throughout its design service life.

Stages in the development and implementation of a SIM system are

a) database development and data acquisition,

b) evaluation,

c) planning,

d) implementation, and

e) verification.

The activities within each stage are not necessarily mutually exclusive and overlap of activities between the various stages does occur.

12.3.2.2 Database development and data acquisition

The database shall consist of all relevant information relating to design, construction and operation of the mooring system — special features and errors and uncertainties in design, fabrication, installation, scheduled and unscheduled inspections, repairs, accidents, modifications, changes in ownership, statutory requirements, monitoring, etc.

Special attention shall be paid to those systems, components and parts thereof already known to develop problems, including

— splash and thrash zones of lines for corrosion and wear,

— wire rope terminations for water ingress, corrosion and fatigue,

— connection hardware for fatigue and corrosion,

— inner layers of lines on drum-type winches for corrosion and wear, and

— lines in the way of fairleads, bend limiters, stoppers and grips for wear and fatigue.

The database shall be stored in a readily retrievable format. A copy of the database shall normally be kept on-board the installation in addition to a master copy kept ashore by the owner.

12.3.2.3 Evaluation

Evaluation shall be structure-specific and site-specific and be based on a fitness-for-purpose philosophy. This shall centre on the intended design service life of the installation, but shall, as a minimum, be reviewed annually as well as following changes in ownership, location, accidents, repairs, modifications and reviews of inspection data.

Evaluation shall involve risk assessment, finite element analysis (FEA), and other forms of assessment as necessary, either of the overall system or parts thereof where damage has arisen or occurred, or of known problem areas, as appropriate. Risk-based inspection approaches can usually be of considerable benefit in the evaluation process and in the scheduling of inspections. Such approaches enable probabilities and risks to be explicitly evaluated and related back to target values.

Where a “safety case” regime is in effect through local regulatory requirements, such safety cases can normally form part of the evaluation.

12.3.2.4 Planning

Planning shall identify the processes, procedures and techniques required to be implemented as a result of the evaluation stage, in order to ensure that the objectives of the fitness-for-purpose assessment are realized. Failure mechanisms, deterioration rates, and the consequences of failure shall be considered so as to determine the methods, frequency and scope of inspections, and possible repair and change-out procedures.

12.3.2.5 Implementation

Implementation refers to the detailed execution of the processes, procedures and techniques identified during planning and should normally include programmes concerned with inspections, maintenance, and monitoring, as well as identifying the need to effect repairs and/or change-outs.

Data gathered during this stage, as well as information issued during the planning stage, should be incorporated into an update of the database, which should be undertaken at least once each year unless justification is presented to extend this period.

12.3.2.6 Verification

Verification of the effective implementation of the SIM system by an independent third party should be considered.

13 Dynamic positioning system

13.1 Basic considerations

13.1.1 General

Dynamic positioning (DP) is a stationkeeping technique consisting of on-board thrusters (and sometimes rudders) that are automatically controlled to maintain a floating structure's position and/or heading. The propulsive force produced by the thrusters/rudders counters the mean and slowly varying actions due to wind, waves and current so as to maintain the structure within pre-set tolerances at a desired point above the sea floor and on a pre-defined heading.

This part of ISO 19901 does not specify DP design requirements. Reference should be made to relevant IMO and RCS publications, see A.13. However, the following subclauses do summarize some pertinent considerations and give other requirements related to DP systems.

13.1.2 DP equipment

A DP system has many components, including thrusters and their ancillary parts, a power supply, distribution systems, and position references. DP equipment redundancy is a major contributor to the DP system's overall reliability. It is common practice to consider subsystems in determining DP reliability, such as

— power subsystem — prime movers and auxiliary equipment, generators, switchboards, associated cabling, etc.,

— thruster subsystem — thrusters and auxiliary equipment, main propellers and rudders, associated cabling, etc.,

— control subsystem — computers and associated software, position reference systems, sensor systems, operator interfaces, power management, associated cabling, etc.

13.2 Design and analysis

13.2.1 Failure modes and effects analysis

Given the potential consequences of a floating structure losing station, DP systems shall be designed to have high reliability and built-in redundancy. Failure modes and effects analysis shall be conducted for floating structures with DP systems and shall be kept up to date during operations. The types of failure to be considered in the FMEA shall include

— the sudden loss of major items of equipment,

— the sudden or sequential loss of several items of equipment with a common link,

— control and monitoring instabilities and failures, and methods of detection and isolation, and

— faults that can be hidden until another fault occurs.

DP systems should be designed so that, as far as is reasonably practicable, there are no common single point failures that could result in loss of position and/or heading. Furthermore, floating structures with DP systems should be assigned an equipment class in accordance with 13.2.2.

13.2.2 DP equipment classes

DP equipment is categorized by redundancy into three equipment classes, whereby class 1 has the least redundancy while class 3 has the most. In this context, “equipment” refers to all equipment described in 13.1.2, which, together with its location/layout on the structure defines the degree of redundancy.

— Equipment class 1 is that where loss of position can occur in the event of a single fault.

— Equipment class 2 is that where loss of position does not occur from a single fault of an active component or system (generators, thrusters, switchboards, remote controlled valves, etc.), and where static components such as cables, pipes and manual valves are adequately protected against accidental damage.

— Equipment class 3 is that where loss of position does not occur as a consequence of one or more of the following events:

  1) single failure of an equipment class 2 active component or system;

  2) failure of any static component or system, whether or not it is protected against accidental damage;

  3) fire or flooding of all components in any one watertight compartment or any one fire subdivision.

Using these classifications and the results obtained from the FMEA, it is possible to allocate a structure with an equipment class notation. The allocation of equipment class notation to a particular floating structure is primarily the responsibility of the structure's flag state authority.

13.3 Design, test and maintenance

Thruster systems should be designed so that, as far as is reasonably possible, there are no common single point failures. A series of sea-trials should be planned in order to verify the thruster system FMEA and, as far as is reasonably practicable, to demonstrate the effects of the various failure modes and ensure that both equipment and procedures are in place to safely cope with failures.

After initial sea trials, an annual survey should be carried out to ensure the DP system has been maintained in good working order. An annual test of all important systems and components should be carried out to document the ability of the DP floating structure to maintain position and heading after single failures associated with the assigned equipment class.

A survey, either general or partial according to circumstances, should be conducted every time a defect is discovered and corrected, or when an accident occurs that affects the safety of the DP floating structure, or whenever a significant repair or alteration is made.

13.4 Operating personnel

Only certified and specially trained personnel shall operate a DP floating structure.

13.5 Determination of stationkeeping capability

A holding capability analysis shall be performed to determine whether the DP system can maintain the position of a floating structure within an acceptable watch circle under ULS and SLS, as appropriate. Such analysis shall be performed both for new designs and for individual operations, see A.13.5.

14 Synthetic fibre rope mooring

14.1 Basic considerations

This clause provides requirements for the design or assessment of permanent and mobile mooring systems incorporating synthetic fibre ropes. It is not intended to cover other marine applications of synthetic fibre ropes such as tanker moorings at piers and harbours, towing hawsers, mooring hawsers at single point moorings, and TLP and SALM tethers. Additionally, very little test data are available for large synthetic fibre ropes permanently deployed around fairleads as such data only address ropes spanning freely between end terminations. Synthetic fibre ropes should not be permanently deployed around fairleads.

All requirements specified in Clauses 5 to 12, as appropriate, shall apply to synthetic fibre rope moorings unless otherwise provided for in this clause.

A cover should be used on fibre ropes to protect against external abrasion expected to occur while in-service, during installation and during recovery. Care should be taken to avoid contact with other ropes, wires, umbilicals, etc., especially during installation, as this could result in damage to the fibre rope.

Fibre ropes shall be qualified in accordance with RCS requirements or other appropriate specifications. Qualification procedures shall address at least

— rope strength,

— tension-elongation properties,

— fatigue resistance;

— rope protection (cover and particle ingress protection), and

— torque properties, as applicable

Synthetic fibre rope mooring technology is rapidly evolving. Designers should take appropriate measures to ensure that their practices incorporate appropriate technological advances.

14.2 Fibre rope mooring analysis

14.2.1 Fibre rope tension-elongation properties

The tension-elongation properties of fibre ropes are non-linear and depend on line tension history. Tension-elongation properties of fibre rope mooring lines have an impact on the total system (floating structure and stationkeeping system) response through

— variability in length of the rope segments over the design life,

— variability in line length and corresponding mean tension under changing environmental conditions, including the effect of the duration of these conditions,

— low frequency motion response of the floating structure, and

— wave frequency line response.

In the absence of a more accurate model, upper bound (storm) and lower bound (post-installation) stiffness values are often used to predict mooring forces and structure offsets, respectively. However, care should be taken in certain circumstances (e.g. VIM-dominated structures), for which the upper bound/lower bound method is not necessarily conservative.

14.2.2 Fibre rope line length

The mooring system shall be designed to maintain sufficient rope clearance from the fairlead and sea floor. The extension at installation and additional set-up and creep elongation during the mooring design service life shall be taken into account in the design, so that the top end of the fibre rope is always clear of the fairlead on the structure, and the minimum tension requirements as defined in 14.5.2 are still met.

The highest point of the installed fibre rope should be at a depth where it is clear of mechanical damage from workboats and surface marine activity, sunlight penetration, salt encrustation and detrimental marine growth.

For permanent and mobile moorings, the lowest point of the installed fibre rope should be at the depth where it is clear of the sea floor in all intact design conditions. If the rope is fitted with a proven particle ingress protection system (e.g. a suitable jacket as specified in ISO 18692), and it can be ensured that the sea floor is free from hard soil areas or other obstructions, contact of the rope with the sea floor may be allowed under installation and redundancy check conditions.

For mobile moorings, fibre ropes with proven particle ingress barriers and jackets may be allowed to contact the sea floor during normal operations if specifically designed for such use. The mooring designer and the rope designer shall address, as a minimum, the following items:

— site survey, including rock outcroppings and soil properties such as abrasiveness and softness;

— damage to jacket and particle ingress barrier during installation due to abrasion with hard soils;

— impact of cyclic motion on soil particle migration through the barrier;

— on-bottom stability for ropes temporarily placed on the sea floor prior to installation;

— inspection issues (see API RP 2I).

14.3 Fatigue analysis

14.3.1 Tension-tension fatigue resistance

The tension-tension fatigue damage shall be evaluated.

The fatigue life of well constructed polyester and HMPE rope lines may typically be assumed to be at least six times that given in Table 3 for a spiral strand wire rope at Q = 0,3 [i.e. K = 1 000 in Equation (12)], for load ranges not exceeding 50 % of MBS.

For other fibre ropes, such as aramid and nylon ropes, fatigue test data are insufficient to develop fatigue design curves. In the absence of better information, the spiral strand curve may be used for the fatigue design of these fibre ropes, see Table 3.

14.3.2 Axial compression fatigue

Axial compression fatigue is not a concern for polyester and HMPE fibre ropes. Axial compression fatigue failure can occur, with some other rope materials, when a rope experiences an excessive number of cycles at low values of the line tension. In order to prevent any part of the rope cross-section from experiencing axial compression, a minimum line tension should be maintained at all times in accordance with 14.5.2.

14.4 Creep analysis

Creep elongation for HMPE ropes shall be predicted for the most critical area of the rope, i.e. the area that is subjected to the highest ambient temperature and highest tension — generally the uppermost area of the rope. For polyester and aramid ropes, creep analysis is not normally required unless they are subjected to unusual actions.

The cumulative creep elongation over the design service life of the rope shall be evaluated where applicable (see 14.2.2), for the mooring system in the intact condition. Where applicable, and particularly for used mooring components, creep elongation from previous operations shall be taken into account.

14.5 Design criteria

14.5.1 Maximum line tension

Maximum line tensions and design safety factors shall be in accordance with 10.2.

14.5.2 Minimum line tension

14.5.2.1 General

The minimum line tension while in-service shall be evaluated as appropriate.

Minimum line tension should be derived by analysis of the leeward mooring lines during design situations. Where needed, the number of cycles of low line tension can be computed by consideration of the long-term distribution of sea states.

The minimum line tension values can be predicted using either frequency or time-domain methods, and non-linearities should be properly incorporated. The effect of time-varying actions on the floating structure should be included since they have an effect on the magnitude of minimum line tensions in leeward lines and on the number of occurrences of minimum tension.

14.5.2.2 Polyester and HMPE

Compression failure is not a concern for polyester and HMPE ropes. However, line integrity should be addressed if the line is not expected to always be in tension while in service.

14.5.2.3 Aramid and other materials

For ropes made of aramids and other higher modulus materials, a minimum line tension of 10 % of MBS should be maintained at all times (except as noted in 14.5.2.4) unless a lower value can be justified by appropriate test. However, if such ropes are subjected to significant twisting, even these minimum tension levels can be insufficient to ensure the integrity of the rope.

14.5.2.4 Minimum mean tensions for pre-deployed lines

Pre-deployed lines made of aramids and other higher modulus materials shall be maintained at a minimum mean tension of 2 % of MBS. However, if such ropes are subjected to significant twisting, even these minimum mean tension levels can be insufficient to ensure the integrity of the rope.

14.5.3 Fatigue

The fatigue safety factor stated in 10.5 shall also be used for synthetic fibre ropes.

14.5.4 Creep elongation

The following requirements do not apply to ropes made of polyester or aramid.

The maximum allowable creep elongation is defined as the lower of

— the extension at which the strength of the rope is 95 % of the original specified MBS, or

— 10 % of the post-installation length after bedding-in,

The predicted creep elongation for the most critical rope area, over the expected service life of the rope, shall be lower than the maximum allowable creep elongation.

14.6 Model testing

For synthetic fibre rope moorings, model tests should account for the non-linear tension-elongation behaviour of the line. If more accurate data or models are not available, tests should at least simulate the lower bound (post-installation) stiffness and upper bound (storm) stiffness values.

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 same numbering system and heading titles have been used in identifying the subclause in the body of this part of ISO 19901 to which it relates.

A.1 Scope

A.1.1 General

Stationkeeping systems for floating structures used for oil and gas applications can be of many types, depending on the characteristics of the structure and on the environmental conditions. Single point moorings are frequently used for ship-shaped floating structures, while spread moorings are used mostly for semi-submersible or other types of structures when maintaining a particular orientation is important. A third type of stationkeeping system is dynamic positioning (DP). Dynamic positioning can be used with either ship-shaped or semi-submersible structures.

Thruster-assisted moorings can be used to reduce mooring line tensions and/or to control heading.

For moorings deployed in ice-infested environments, additional information on ice-induced actions should be referenced in ISO 19906 [5].

A.1.2 Spread moorings (catenary, taut-line and semi-taut-line)

Figure A.1 is an illustration of a fairly typical catenary spread moored semi-submersible used for drilling operations. For floating production structures, spread moorings are often used with semi-submersibles. Since the environmental actions on a semi-submersible are relatively insensitive to direction, a spread mooring system can adequately hold the structure on location. This solution can also be used with ship-shaped structures, which are more sensitive to the directionality of the environmental actions, when the prevailing weather at the site comes from one direction, so that the structure can be oriented with the narrow dimension into the weather. Spread moorings can incorporate chain, wire rope, synthetic fibre rope, or a combination of the three. Drag anchors or anchor piles are generally used to terminate the mooring lines.

Alternative spread mooring systems include taut or semi-taut-line systems.

The main advantage of a spread mooring system is that it fixes the orientation of the floating structure, so that drilling, completion and well intervention operations can be carried out on subsea wells located immediately below the structure. On the other hand, a spread mooring system has a fairly large mooring spread (several times the water depth). The presence of anchors and mooring lines should be considered in the installation or maintenance of pipelines, risers or any other subsea equipment.

A.1.3 Single point moorings

Single point moorings are used primarily for ship-shaped floating structures such as FPSOs and FSOs. Their main characteristic is that they allow the structure to weathervane. There is wide variety in the design of single point moorings but they all perform essentially the same function. Single point moorings interface with the production riser and the structure. A brief summary of typical single point mooring systems is as follows.

a) Turret mooring

In this type of mooring system, catenary mooring lines are attached to a turret, which is essentially part of the structure to be moored. The turret includes bearings to allow the structure to rotate (yaw) independently of the mooring system.

The turret can be mounted externally from the structure's bow or stern with appropriate reinforcements (Figure A.2) or internally (Figure A.3). The chain table can be above or below the waterline. Flow from the turret into the process facilities is via marine hoses or flexible pipes that extend upward from the sea floor to the bottom of the turret.

In some cases the turret is designed such that the lower chain table can be disconnected to enable the floating structure (usually self-powered) to depart from the location in advance of a foreseeable severe environmental event, e.g. a tropical cyclone or an approaching iceberg. After disconnection, the self-buoyant chain table remains submerged at a pre-set depth while supporting the mooring lines and risers.

A variant of this arrangement is a buoyant submerged turret assembly designed for easy connection and disconnection in order to temporarily moor specially modified export tankers for direct loading of produced oil. This arrangement is also used for permanent floating structures (FPSO or FSO). In this design the turret assembly contains the main bearing that allows weathervaning of the tanker or floating structure.

b) Catenary anchor leg mooring (CALM)

A CALM system consists of a large buoy that supports a number of catenary mooring legs anchored to the sea floor (Figure A.4). Such systems are typically used as export terminals for direct loading of tankers at the production field. Riser systems or flow lines that emerge from the sea floor are attached to the underside of the CALM buoy. A hawser, typically a synthetic rope, connects the tanker to the buoy. Since the response of the CALM buoy to environmental actions is totally different from that of the tanker, this system is limited in its ability to withstand environmental conditions. When sea states attain a certain magnitude it is necessary to disconnect the tanker.

CALMs have also been used to moor FPSOs and FSOs. In order to overcome the limitation noted above, rigid structural yokes with articulations are used in some designs to tie the floating structure to the top of the buoy. An example is shown in Figure A.5. This rigid articulation virtually eliminates horizontal motions between the buoy and the tanker. A number of variations on this basic arrangement are in use.

c) Single anchor leg mooring (SALM)

This system, illustrated in Figure A.6, employs a vertical chain riser system that is pre-tensioned by a surface piercing buoy. The buoyancy acting on the top of the riser tends to restore the riser to the vertical position (inverted pendulum effect).

A tanker can be moored to the top of this SALM buoy with a hawser. The base of the riser is usually attached through a U-joint to a piled or deadweight concrete or steel structure on the sea floor. In deeper water, the chain riser system can be replaced by a tubular riser structure. Variants of this concept have been used to moor a floating structure (FPSO or FSO) using a rigid arm.

A.1.4 Dynamic positioning (DP) systems

DP is a technique of automatically maintaining the position of a floating structure within a specified tolerance by controlling onboard thrusters which generate thrust vectors to counter the wind, wave and current actions. DP is particularly well suited for a floating structure designed to arrive and leave location frequently, such as an extended well test system.

The DP referred to in this part of ISO 19901 describes stationkeeping without moorings.

A.1.5 Thruster-assisted moorings

Many floating structures designed to operate with moorings are also equipped with thrusters and thruster control systems. The thrusters can be used to control the structure's heading, reduce mooring forces under severe environment, or increase the workability of the floating structure.

A.1.6 Permanent and mobile stationkeeping systems

A.1.6.1 Definitions

Stationkeeping systems used for production operations with long design service lives are generally defined as permanent. The mooring for a floating production system (FPS), for example, is usually a permanent mooring since FPSs typically have design service lives of over 10 years. Mobile stationkeeping systems are usually deployed on one location for only a relatively short period of time. Examples of mobile systems include those for MODUs, and for tenders moored next to another platform such as floatels, drilling tenders, barges, and service or construction vessels. The division between mobile and permanent moorings is sometimes not clear for operations with service lives of a few years. In this case, the owner should make a judgement based on the risk of exposure to severe environments and the consequences of a mooring failure.

A.1.6.2 Mooring hardware

Mobile moorings typically incorporate hardware components that can be rapidly deployed and retrieved. This requirement does not apply to permanent moorings. Many mooring components such as anchor piles, linear winches, buoys and chain jacks, which are often not suitable for mobile moorings, can be used for permanent moorings.

A.1.6.3 Installation

The deployment of a mobile mooring system is normally carried out with the assistance of work boats. This operation is simple and usually takes no more than a few days. The deployment of a permanent mooring system often requires the assistance of much heavier installation equipment, such as a derrick barge or a purpose-built work boat. A portion of the mooring is usually pre-installed.

A.1.6.4 Inspection and maintenance

A mobile mooring system can often be visually inspected during retrieval or deployment. Retrieving a permanent mooring system for inspection can be very expensive, so the inspection of these mooring systems involves the use of divers or remotely operated vehicles (ROVs). Replacing faulty mooring components is much easier for mobile than for permanent mooring systems.

A.1.7 Mooring line components

A.1.7.1 General

Mooring lines for floating structures are usually made up of wire rope, chain, synthetic fibre rope or a combination thereof. Many possible combinations of line type, size and location, and size of clump weights or spring buoys can be used to achieve the required mooring performance.

A.1.7.2 Wire rope

Being much lighter than chain, wire rope provides a greater restoring force for a given pre-tension. This becomes increasingly important as water depth increases. However, to prevent anchor uplift with an all-wire rope system, much longer line length is required. A disadvantage of an all-wire rope mooring system is wear due to long-term abrasion where it contacts the sea floor. For these reasons, all-wire rope mooring systems are seldom used for permanent moorings.

A.1.7.3 Chain

Chain has shown durability in offshore operations. It has better resistance to bottom abrasion and contributes significantly to anchor holding capacity. However, in deep water an all-chain system imposes an increasing penalty on the floating structure’s payload capacity because of its weight and pre-tension requirements.

A.1.7.4 Synthetic rope

Synthetic fibre ropes made of polyester, HMPE or aramid fibres are increasingly being used in deepwater mooring systems due to their much lighter submerged weight and greater elasticity compared with steel wire rope. Synthetic fibre ropes also have very long fatigue lives compared with steel ropes (see A.14).

A.1.7.5 Chain/rope combinations

In these systems, a length of chain is typically connected to the anchor. This provides good abrasion resistance where the mooring line contacts the sea floor and its weight contributes to anchor holding capacity. The choice of chain or wire rope at the structure's end and the type of termination depends on the requirements for adjustment of line tensions during operations. By proper selection of the lengths of wire rope and chain, a combination system offers the advantages of reduced pre-tension requirements with higher restoring force, improved anchor holding capacity and good resistance to bottom abrasion. These advantages make combination systems attractive for deepwater mooring.

A.1.7.6 Clump weight

Clump weights are sometimes incorporated in mooring lines to optimize performance. By providing a concentrated weight to the mooring line at a point close to the sea floor, a clump weight can be used to replace a portion of chain and increase the restoring force of a mooring line. Using clump weights in a mooring line design requires consideration of potentially adverse effects, such as increased use of connecting hardware, installation complexity, undesirable dynamic response and increased risk of embedment in the seabed.

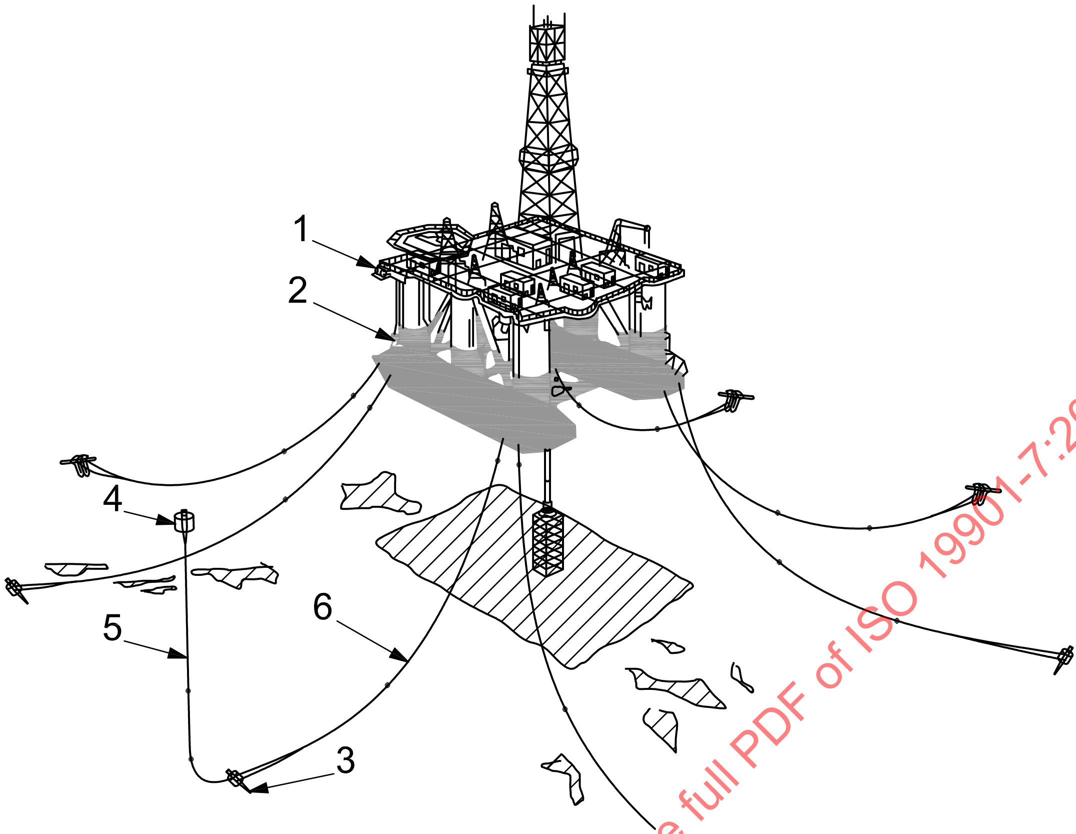
A.1.7.7 Spring buoy

Spring buoys are surface or sub-surface buoys that are connected to a mooring line. The benefits of spring buoys are

- reduced weight of mooring lines to be supported by the structure or MODU hull — particularly advantageous for semi-submersibles moored in deep water,
- reduced effects of line dynamics in deep water, and
- reduced floating structure offset for a given line size and pre-tension.

The adverse effects of spring buoys are

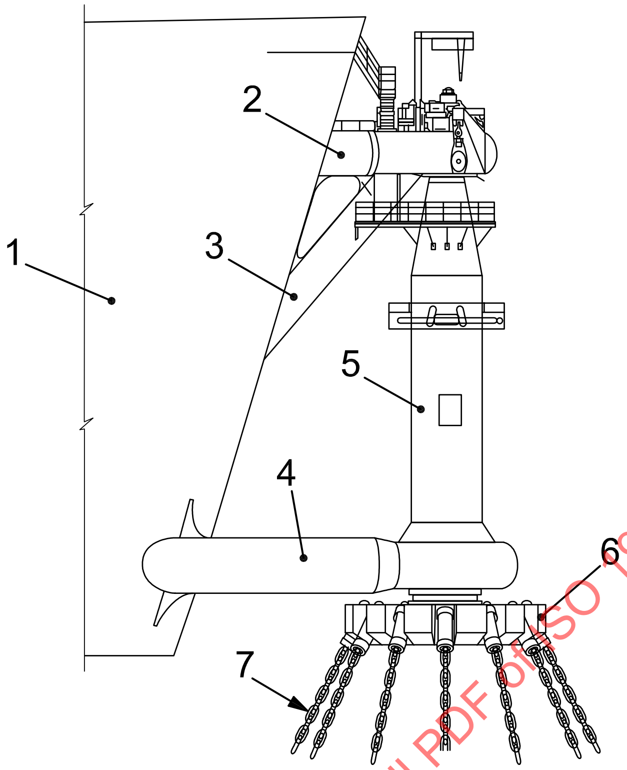
- increased use of connecting hardware, and installation complexity, and
- potential for increased environmental actions on the mooring lines due to the dynamic response of the buoys in heavy seas.



Key

1 winch or windlass
2 fairlead
3 anchor
4 pendant buoy
5 pendant line
6 mooring line

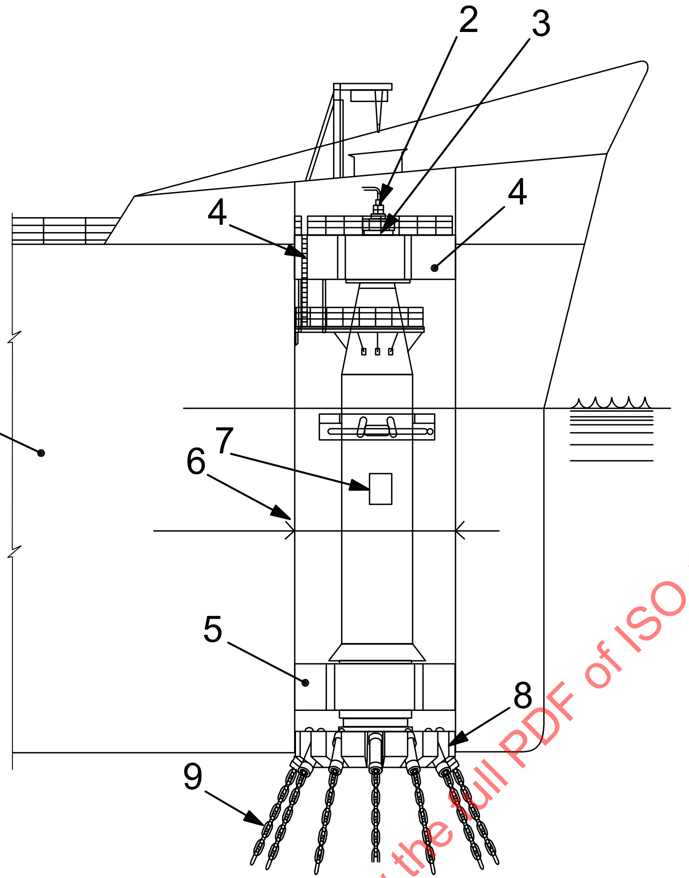
Figure A.1 — Spread mooring



Key

1 floating storage unit (FSU)
2 upper connection structure
3 diagonal brace structure
4 lower connection structure
5 vertical turret shaft
6 chain table
7 mooring chain (typical)

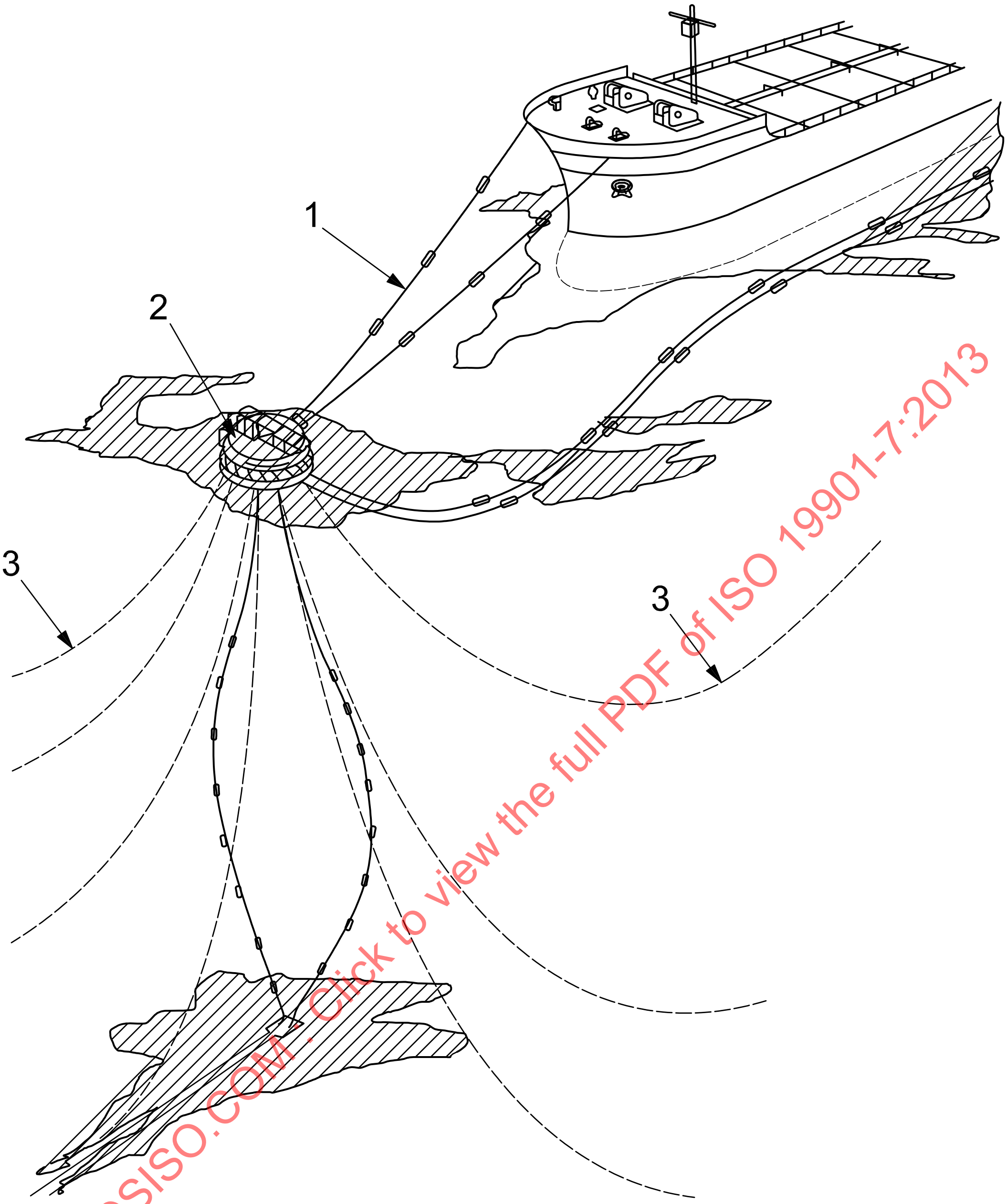
Figure A.2 — External turret mooring system



Key

1 floating storage unit (FSU)
2 in-line swivel
3 toroidal swivel
4 upper connection structure
5 lower connection structure
6 turret well wall
7 vertical turret shaft
8 chain table
9 mooring chain (typical)

Figure A.3 — Internal turret mooring system



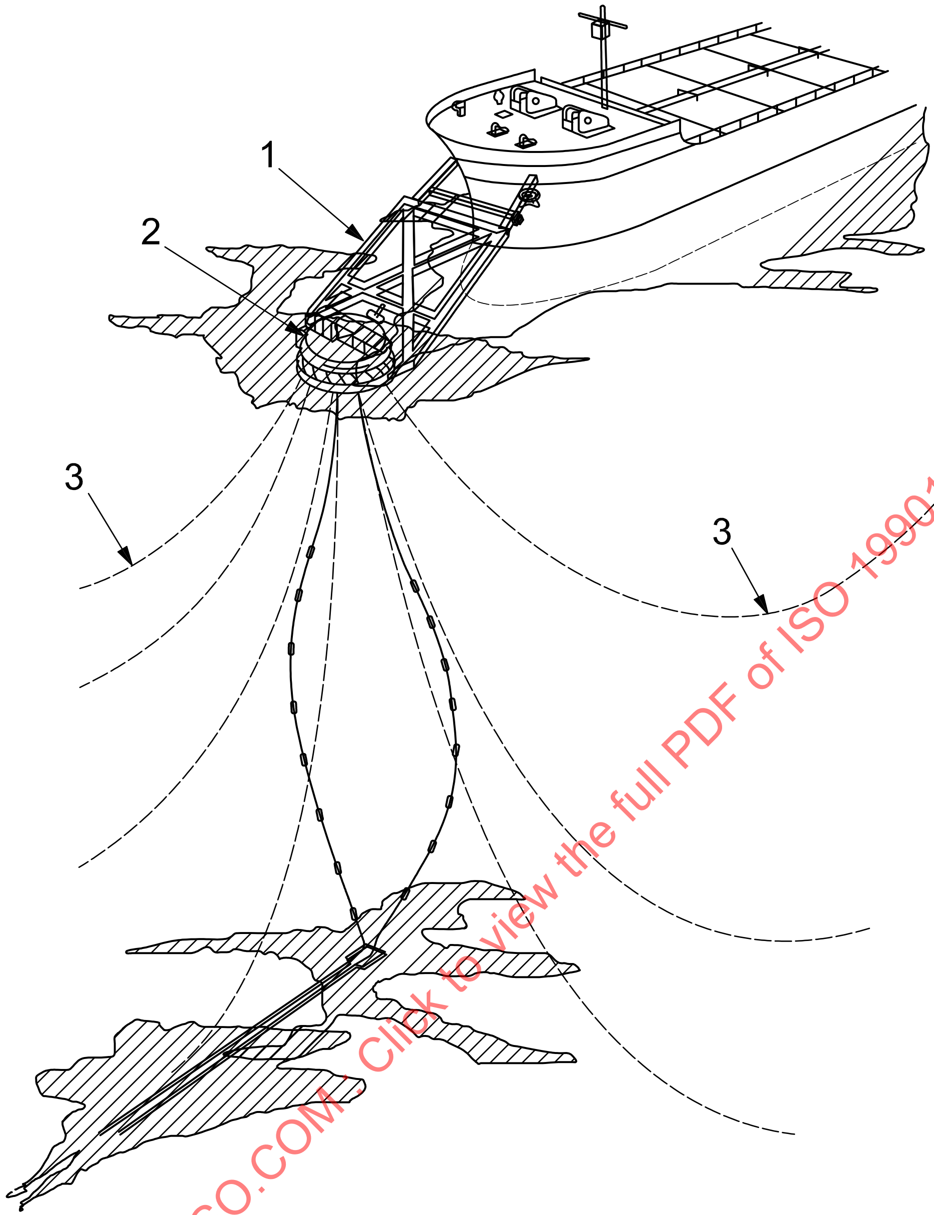
Key

1 hawser

2 buoy

3 mooring line

Figure A.4 — Catenary anchor leg mooring (CALM) system with hawsers



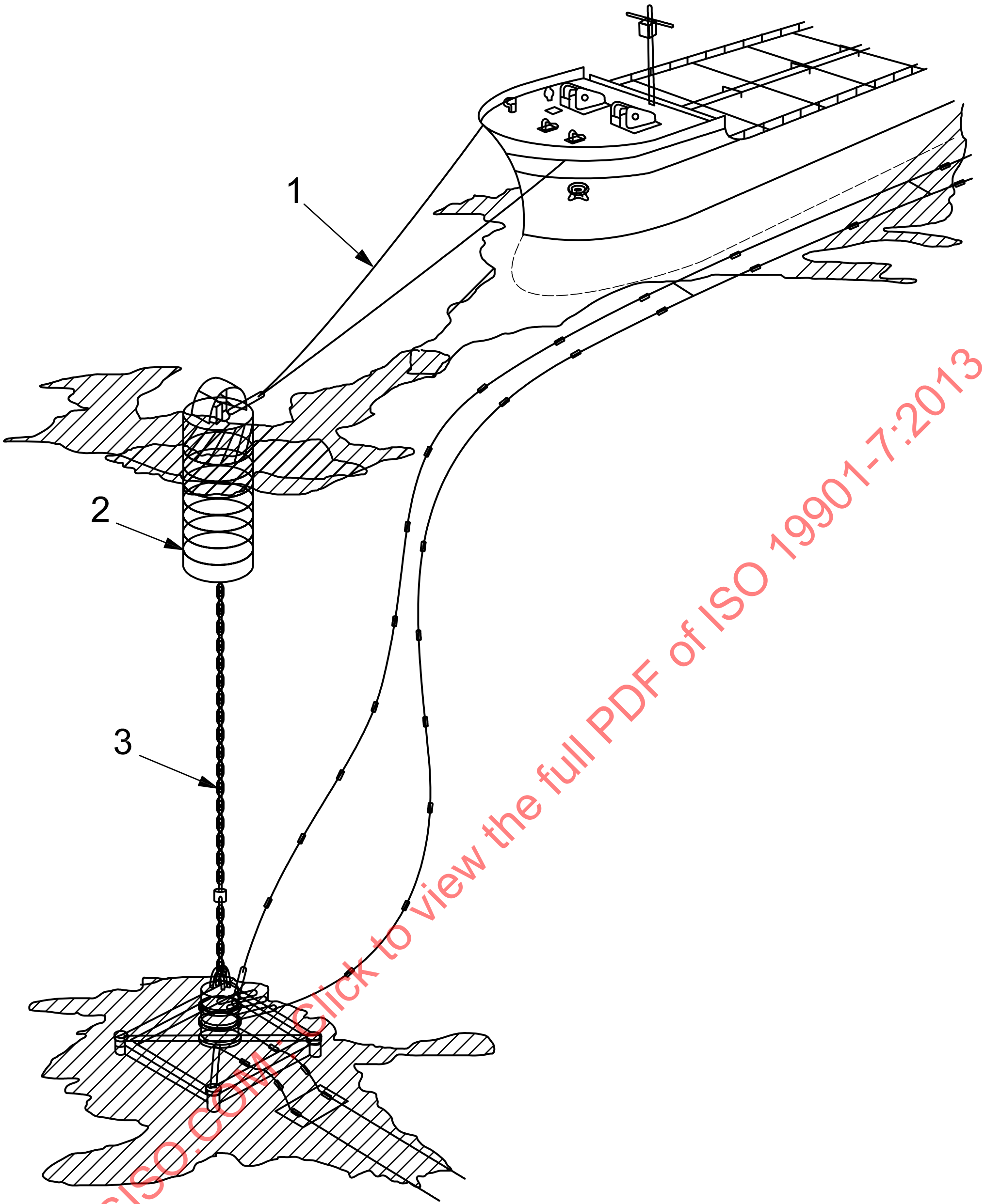
Key

1 yoke

2 buoy

3 mooring line

Figure A.5 — Catenary anchor leg mooring (CALM) system with fixed yoke



Key

1 hawser
2 buoy
3 chain riser

Figure A.6 — Single anchor leg mooring (SALM) system with vertical chain riser system, pre-tensioned by a surface piercing buoy

A.2 Normative references

No guidance is offered.

A.3 Terms and definitions

No guidance is offered.

A.4 Symbols and abbreviated terms

No guidance is offered.

A.5 Overall considerations

A.5.1 Functional requirements

A.5.1.1 Riser considerations

Risers transfer fluids between the sea floor and the floating production system or MODU, and constitute one of the primary design constraints for the mooring system. The riser system often places limitations on the allowable offset. In the event of excessive structure offset, mooring line adjustments such as slackening the leeward lines is sometimes performed to avoid damage to the riser system. An equally important consideration is interference between mooring lines and risers during both operational and extreme weather conditions. The mooring system and riser system should therefore be designed to accommodate each other and coordination of these two design efforts is essential.

A.5.1.2 Subsea equipment considerations

Subsea equipment such as templates, riser bases, satellite wells and flowlines should be located clear of any potential mooring line interference. Any contact between mooring lines and subsea equipment during installation, operation or maintenance presents a high potential for damage to both the equipment and the mooring lines. If interference, or the potential for interference, appears unavoidable it is sometimes possible to alter the layout and design of the mooring system through the use of an asymmetric mooring line arrangement, or the use of clump weights or spring buoys. Coordination of the mooring system design with the subsea equipment layout is essential.

A.5.2 Safety requirements

A.5.2.1 General regulations

Regulations can be international regulations, requirements specified by the flag state or by the owner. In addition, a coastal state and/or port authority can have its own set of regulations concerning offshore operations. The owner shall comply with all applicable rules, depending upon the floating structure's location and on the type of operations to be conducted.

A.5.2.2 Stationkeeping systems hazards

This subclause outlines a number of hazards relating to potential mooring system failures. Their importance should be recognized by mooring system designers and owners who should seek to minimize their occurrences and mitigate their consequences.

a) **Rapid disconnect mooring release mechanism malfunction**

In cases where a rapid disconnect system is adopted, hazards arise if the systems do not activate on demand, or are activated when not required to do so. Rigorous procedures should therefore be implemented to ensure correct operation. Planned maintenance and testing of such systems should be practised, providing mooring integrity is not compromised in so doing.

b) **Failure of structures supporting anchoring equipment, fairleads and winches**

Hazards to the structure can be caused by poor design and construction, and by in-service demands resulting in equipment failure from external actions, vibration, corrosion and wear. In extreme cases, watertight integrity can be affected. Thus, a rigorous planned maintenance schedule, as part of a structural integrity management (SIM) system, should include regular inspections of all mooring component foundations and internal supporting structure. Maintenance should also cover the correct operation of all moveable parts and protection from corrosion.

c) **Manufacturing defects and processes**

Wire rope and chain, synthetic fibre rope, common links, connectors, chasing equipment, pennants, etc., are all subject to imperfections in the manufacturing process. Such equipment should only be accepted on the basis of quality control procedures, testing and approval and constructed to recognized industry standards.

d) **Mechanical, electrical, and hydraulic failures relating to mooring systems**

Hazards from these supply systems can lead to the failure of control, reference or sensors, action response and loss of manual over-ride. Mechanical failures can occur to windlass brakes, clutches and sleeves, pawls, gears, splines, cracked discs or drums, cooling, striker bar, wire spooling and tensioning devices, etc. Hydraulic failures can occur to pipework, seals, joints, pumps, brake valves, greasing systems, oil contamination, leaks and levels, or emergency release valves. Electrical failures generally affect control and monitoring of position, tension and power systems. Regular inspection and planned maintenance procedures should be set in-place to minimize the likelihood of failure of these systems.

e) **Mooring system overload, fatigue and insufficient anchor holding capacity**

Overload is defined as any tension in excess of a predefined limit or of the capacity of an anchor. Either event can cause a loss of position. Hazards include inadequate use of propulsion, exceptional environmental conditions, inappropriate anchor penetration, excessive tensioning, equipment failure, and incorrect installation and retrieval operations. Excessive wear can arise in chain at the touchdown point when the chain frequently alternates between very slack and tensioned condition to such an extent that chain links fall onto each other. Written operating procedures for managing mooring line tensions should be clearly defined and should be readily available to operating personnel. This includes the use of thrusters, if available, and the redistribution of line tensions to prevent design limit exceedance of any component.

Anchor holding failure sometimes occurs through overloading of the soil foundation, inappropriate equipment or anchor design for the soil conditions (e.g. fluke angle), equipment failure (including shackles) or inadequate line lengths for extreme conditions. The consequence of these items is likely to be anchor drag.

In some areas of the world the practice on receipt of severe weather forecasts is to reduce tensions on all mooring lines and to evacuate personnel. In other areas, personnel stay on-board and initiate measures to protect the installation. Clear operating procedures should be developed for the selected scenarios.

The adequacy of the mooring system to resist cyclic actions (fatigue) is usually investigated and proved through a systematic mooring analysis. Practical measures can be adopted to reposition the sections of mooring lines subjected to concentrated fatigue excitation, e.g. at the fairleads or at the touchdown point.

All components should be maintained to a satisfactory level achieved by the use of a SIM system with unambiguous criteria for discard and replacement of key components.

f) In-service degradation of mooring components

Mooring line integrity is generally degraded by corrosion, wear and damage. Corrosion is a major concern, especially in the splash zone, for chain and wire systems. If a high corrosion potential exists between components of a mooring system, hydrogen levels can then be sufficient to cause embrittlement in any high strength materials present.

Abrasion or wear in wire ropes generally occurs at the winch, fairlead and touch-down point. Normal operations will ensure shifting of the contact areas. Abrasion in chain will be concentrated at link contact points and any unforeseen contact with the floating structure.

Damage during deployment and retrieval is possible unless careful procedures are followed. For example, wire ropes are particularly subject to crushing damage on winches from high tension spooling. The spiral structure of wire rope can cause a torque build-up if dragged along the sea floor resulting in a hockle or loop should the rope tension reduce. Also, the improper use of chasers during wire rope retrieval can cause damage to the rope and connectors.

A regular inspection programme is essential to monitor the integrity of the moorings.

g) Inadequate operating, maintenance and handling procedures

Operating procedures are developed with the objective to minimize the risk of mooring failure; these should adequately cover handling during deployment and retrieval, inspection/discard criteria or maintenance requirements. Permanent moorings are less likely to suffer from handling damage as they generally remain in-place for longer periods. Handling procedures should cover normal handling operations aimed at minimizing service wear, scuffing, abrasion, chaser damage, running operations, winching wire onto drums or chain into a locker.

The condition of all mooring lines will deteriorate with time in service, and adequate inspection and maintenance programmes will be necessary to ensure continuous integrity.

h) Operator error

Hazards arising from operator error include the inability to implement all items described in this subclause and as defined in the operating procedures. Operator errors can be minimized with adequate training of responsible personnel. This includes attendance at appropriate training courses, the provision of clear procedural guides and operations manuals, setting out the chain of command, and drills and briefings before undertaking work.

A.5.3 Planning requirements

No guidance is offered.

A.5.4 Inspection and maintenance requirements

No guidance is offered.

A.5.5 Analytical tools

No guidance is offered.

A.6 Design requirements

A.6.1 Exposure levels

The concept of exposure levels was initially introduced for fixed steel structures in ISO 19902 [4]. Exposure levels for floating structures are fully defined in ISO 19904-1.

A.6.2 Limit states

A.6.2.1 General

No guidance is offered.

A.6.2.2 Limit states for stationkeeping systems

A.6.2.2.1 Ultimate limit states

These limit states correspond to the stationkeeping system's resistance to most probable maximum actions, such as those arising from the design environmental events.

A.6.2.2.2 Serviceability limit states

These limit states are related to the criteria governing normal functional use of the floating structure. The stationkeeping design should comply with the serviceability requirements of the floating structure, risers, drilling equipment, production facilities, etc., as defined by the owner.

As an example, a SLS is reached when the action effects on the floating structure are such that the intended operations on the floating unit (such as drilling, producing, maintaining gangway connections, etc.) can no longer be carried out.

Alternatively, if the owner does not specify the serviceability requirements, the designer should establish the limiting operating environment. This should become part of the operations manual, and should be known to the people responsible for the drilling and well intervention or production operations in order that timely plans to suspend operations can be performed.

A.6.2.2.3 Fatigue limit states

Fatigue limit states for stationkeeping systems refer to cumulative damage in the system components due to environmental cyclical action effects.

A.6.2.2.4 Accidental limit states

No guidance is offered.

A.6.3 Defining design situations

No guidance is offered.

A.6.4 Design situations

A.6.4.1 General

No guidance is offered.

A.6.4.2 Design situations for ULS

A.6.4.2.1 General

When a risk analysis is performed, it should include historical experience, the design service life and intended use of the stationkeeping system, the operating personnel safety assessment, environmental damage prevention, the probability of mooring damage or loss when subjected to environmental conditions defined by parameters with various return periods, and the financial loss due to mooring failure.

Calibration of ULS safety factors is discussed in References [35] and [47].

A.6.4.2.2 Permanent moorings

Permanent mooring systems should be designed for the combination of wind, wave, and current conditions that are likely to induce extreme values of action effects. In practice, this is often approximated by the use of multiple sets of design situations. For example, in the case of a 100 year return period, three types of design situations are often investigated:

— the 100 year return period waves with associated wind and current;

— the 100 year return period wind with associated waves and current;

— the 100 year return period current with associated waves and wind.

The directional combination of wind, waves and current that results in the most severe effects should be used to verify the design of the permanent installation being considered, consistent with the site's environmental conditions.

Selection of design situations requires careful consideration. For example, large ship-shaped structures are dominated by low-frequency motions. Since low-frequency motions generally increase with decreasing wave periods, the 100 year return period waves do not necessarily yield the most severe action effects on moorings. Smaller, higher frequency waves with shorter return periods could yield larger low-frequency motions and thus higher actions on the mooring system.

The possibility of minor damage to the mooring system, floating structure and related systems can be acceptable as a consequence of an emergency disconnection, at the discretion of the owner. However, an emergency disconnection should not significantly increase the risk to personnel, the environment, or other structures in proximity to the floating structure or its mooring system.

A.6.4.2.3 Mobile moorings

A.6.4.2.3.1 Mobile moorings for structures not in proximity to other installations

The minimum return period in some jurisdictions is fifty years.

A.6.4.2.3.2 Mobile moorings for structures in proximity to other installations

An example of operations in proximity to other structures is a MODU with mooring lines deployed over a pipeline. Damage to the pipeline can occur if the anchors are dragged onto the pipeline. Other examples include a drilling and well intervention tender, a floater, and a service vessel moored next to a structure.

A.6.4.2.3.3 Mobile moorings redundancy check condition

No guidance is offered.

A.6.4.3 Design situations for SLS

Generally, design situations for SLS are less severe than those for ULS. However, in some FPS operations, where the stationkeeping system is intended to allow the floating structure to continue production during a severe storm, the design situations for SLS can be the same as those for ULS.

A.6.4.4 Design situations for FLS

Calibration of FLS safety factors is discussed in [35] and [60].

A.6.4.5 Design situations for ALS

Calibration of ALS safety factors is discussed in References [35] and [68].

A.7 Actions

A.7.1 General

No guidance is offered.

A.7.2 Site-specific data requirements

A.7.2.1 Data collection and analysis

For further guidance on data collection and analysis, see ISO 19901-1, and relevant RCS rules, for example Reference [28].

For example, operations in tropical cyclone areas, such as the Gulf of Mexico and the South China Sea, are characterized by generally mild environments with well defined severe storms during the cyclone season (hurricanes in the Gulf of Mexico, typhoons in the South China Sea). In these areas, for operations out of the cyclone season, the environmental definition of the design situation may be determined from an analysis of available environmental data excluding tropical cyclones.

A minimum wind speed associated with tropical cyclone conditions has been specified in order to avoid misinterpretation of sparse statistics, see 6.4.2.3.1.

A.7.2.2 Water depth

No guidance is offered.

A.7.2.3 Soil and sea floor conditions

A.7.2.3.1 General

General geotechnical requirements for offshore structures can be found in ISO 19901-4 [3].

A.7.2.3.2 Permanent moorings

A.7.2.3.2.1 General

The areal extent of the foundation system for floating structures greatly exceeds that of fixed structures and TLPs. Requirements for site investigations should be guided primarily by the type of platform to be installed, the availability and quality of data from prior site surveys, and the consequences that would result from a partial or complete foundation failure.

It is recommended that a high-quality, high-resolution geophysical survey be performed over the entire areal extent of the foundation. This survey should be subjected to a realistic geological interpretation and the results should be integrated with existing geotechnical data (if any) to assess constraints imposed on the design by geological features. Such an integrated study can then serve as a guide to develop a scope of work for the vertical and horizontal extent of the final geotechnical investigation (i.e. number, depth, and location of soil borings and/or in-situ tests such as PCPTs, i.e. Cone Penetrometer Tests equipped with pore pressure transducers) and to aid in the interpretation of the acquired geotechnical data. Previous site investigations and experience can permit a less extensive site investigation. Some examples of these integrated geoscience studies are given in References [90] and [91].

A.7.2.3.2.2 Soil sampling and laboratory testing

Should the designer choose to rely on soil sampling and laboratory testing instead of in-situ testing, the designer should be aware that the measured properties of soil samples retrieved from deep waters can differ from in-situ values. Without special precautions, the relief of hydrostatic pore pressure and its resulting effect on any dissolved gases can yield soil properties significantly different from those applicable to in-situ conditions. Because of these effects, in-situ or special laboratory testing to determine soil properties is preferred. Some of the geotechnical tools available when rotary drilling techniques are employed for deep water investigations are discussed in Reference [92]. Coring with “jumbo” or “long” coring devices has also been shown to provide shear strengths equivalent to those obtained by rotary drilling methods and holds promise as an alternative coring method (see References [93] and [94]).

A.7.2.3.2.3 In-situ testing

In-situ testing can provide a more reliable estimate of soil parameters and alleviate issues with sample disturbance. Typical tools used include: the remote vane (either seabed or downhole units), the piezoprobe (to obtain estimates of in-situ pore pressure and permeability), and PCPT. Advantages of the PCPT method include obtaining a continuous profile of soil resistance that allows for a detailed stratigraphic assessment. However, CPT results should be calibrated against results from other in-situ tests (e.g. in-situ vane) and laboratory tests, as available, in order to quantify soil resistance. A comprehensive discussion of PCPT data interpretation can be found in Reference [95]. Other promising tools include the T-bar penetrometer [96].

A.7.2.3.2.4 Recommended sequence for site characterization

A site investigation programme should be performed for each platform location. The programme should, as a minimum and preferably in the order listed, consist of the following:

a) Background geophysical survey

Regional geological data should first be obtained to provide information of a regional character, which can affect the analysis, design and siting of the foundation. Such data should be used in planning high-resolution surveys and geotechnical site investigations, and to ensure that the findings of the subsurface investigation are consistent with known geological conditions. Site-specific background data should include a re-examination of the 3-D, multichannel data obtained for exploratory purposes and a review of the geohazard study used to site the exploratory wells. The 3-D data set can be re-processed to enhance its high frequency content. Suggested reading for further information is given in [97].

b) Sea floor and sub-bottom survey

Site-specific, high-resolution geophysical information should be obtained relating to the conditions existing at and near the surface of the sea floor. The survey should include the mapping and description of all sea floor and sub-bottom features that can affect the foundation system. Such features include: sea floor contours, seabed slope angles, shallow stratigraphy, position of bottom shapes which could affect scour, boulders, obstructions and small craters, fluid expulsion features, pockmarks, shallow faults, slump blocks, drill cuttings, previous usage of sea floor, and gas hydrates.

The survey should use geophysical equipment and practices appropriate to the water depth of interest and provide high-resolution imaging of the sea floor as well as detailed stratigraphic information to a reasonable penetration below the zone of influence of the structure. The stratigraphic data thus obtained should be integrated with the geotechnical data collected subsequently (see next item) to allow for soil data interpolation and/or extrapolation in the event the anchor locations are shifted after geophysical and geotechnical surveys.

c) **Geotechnical investigation**

The sampling and in-situ testing intervals should ensure that each significant stratigraphic layer is properly characterized. The design soil parameters for various soil strata should be determined from a field programme that tests the soil in as nearly an undisturbed state as feasible. Because the quality of soil samples can be expected to decrease with increasing water depth, the use of in-situ testing techniques is encouraged for deepwater sites. In addition, soil samples can be required to provide advanced engineering soil properties.

The content and scope of a deepwater soil investigation should always be tailored to the project-specific conditions. When planning a soil investigation the following general recommendations are given.

— If no previous experience is available for the site, the minimum scope should consist of one boring with alternating sampling and down-hole PCPTs at two of the anchor clusters.

— If these boreholes show great vertical and/or lateral variability across the mooring pattern increasing the number of borings should be considered.

— The laboratory investigations should comprise standard classification tests and determination of the undrained shear strength of clay samples. In addition; for anchor piles, drag embedment anchors and plate anchors, consolidated, constant volume Direct Simple Shear (DSS) tests and/or Unconsolidated Undrained (UU) triaxial tests are preferred; for suction anchors consolidated undrained compression and extension triaxial tests are also advisable.

— A few thixotropy tests to provide a basis for assessment of the set-up versus time after anchor installation.

— Correlate the PCPT net cone resistance with the DSS and/or UU triaxial test undrained shear strength values and use the derived bearing capacity factor(s) to develop continuous undrained shear strength profiles over the depths covered by the PCPTs.

— A few consolidation tests to determine the over-consolidation ratio (OCR) of clay layers generally improves the basis for derivation of the characteristic undrained shear strength profile for the location.

— If deep deposits of clay are encountered, remote vane tests should be considered in combination with the above scope bearing in mind that the vane strengths should be corrected for strain-rate effects before being used in design [147].

However, if high-quality geotechnical data already exist in the general vicinity of the anchor pattern and little variation of soil properties is inferred over the areal extent of the foundation, or if extensive experience with the chosen foundation concept in the area can be drawn upon, the above recommendations can be modified as appropriate (see References [98] and [99]).

The minimum vertical extent of the site investigation should be related to the expected zone of influence of the actions imposed by the base of the foundation and should exceed the anticipated design penetration by at least the anchor diameter or anchor fluke width, B (see Figure A.26). If Reverse End Bearing (REB) at the suction anchor tip is taken into account in the vertical capacity analysis, soil characterization up to three diameters for suction piles or three fluke widths for plate anchors below the design penetration depth can be more appropriate. It is critical to ensure that no high-permeability layers are present within the zone influenced by the mobilization of the REB, particularly if the anchor is expected to resist long-duration forces such as those imposed by loop currents.

If the soil investigation is performed primarily using PCPT, it is recommended that at least one boring and/or long core be taken to properly calibrate the PCPT results. This boring/core should be taken at one of the PCPT locations.

The site investigation should also consider that during the detailed platform and mooring design process, the seabed location of the anchors can change due to changes in mooring line lengths and/or headings, field layout, platform properties, and mooring leg properties.

Some examples of the scope of deepwater investigations are given in References [98] and [100] and examples of data interpretation are given in References [101] and [102].

d) Soil testing programme

The main goal of the laboratory testing programme should be to properly evaluate all input parameters required for geotechnical and structural design, for all significant strata. When applicable, testing should be performed in accordance with recognized standards (i.e. ASTM or others).

Additional testing should be performed to define the creep and cyclic behaviour of the soil to allow prediction of soil structure interaction due to sustained and cyclic loading. Consideration should be given to the performance of permeability, thixotropy and consolidation tests in order to understand set-up effects for driven piled structures and capacity consideration for suction piles and suction caissons.

In all-clay soils, the site investigation and laboratory testing programme should provide the following information needed for the reliable design of pile and plate anchors, as applicable for the type of anchor size, and anchor loading.

— General soil description, classification, and index testing.

— Soil stress history and OCR, soil compressibility (i.e. unload and reload moduli), as measured in Constant Rate of Strain (CRS) tests or constant load tests.

— Soil permeability.

— Remoulded shear strength and soil sensitivity.

— Monotonic and cyclic shear strength under appropriate average and cyclic stresses for triaxial compression, extension, and DSS stress paths; samples should preferably be anisotropically consolidated and cyclic tests should preferably be performed at the expected action period in addition to the normal cycle period of 10 seconds.

— Creep data to define possible loss of shear strength under sustained actions (in cases where large sustained actions, e.g. loop currents, are important). Cyclic stresses should be superimposed on the sustained stresses if relevant for the actual design situations.

— Remoulded soil consolidation characteristics (compressibility and permeability).

— Reconsolidated remoulded soil strength characteristics.

— Soil thixotropy.

— Parameters needed for generation of p-y curves (i.e. 50 % strain factor, ε_{50})

Databases for cyclic soil properties are available in References [100] and [103] for Gulf of Mexico clays. Such databases should be used to interpret tests results and reduce the number of site-specific cyclic tests.

e) Additional studies

As applicable, additional analytical studies or scaled tests should be performed to assess the following aspects:

— scouring potential;

— earthquake ground response studies or analysis;

— sea floor instabilities in the area where the foundation systems are expected to be placed;

— set-up effects.

A.7.2.3.3 Mobile moorings

Anchors for MODUs are often designed without site-specific soil data. Although this approach can often be justifiable, appropriate fit-for-purpose site specific soil investigations are generally preferred where sea floor characteristics differ from established experience or detailed anchor analytical assessment is needed. See guidance in A.7.2.3.1.4 for planning a soil investigation.

A.7.2.4 Wave statistics

Because of the random nature of the sea surface elevation, stationary statistical records of water surface elevation (sea states) are usually described in terms of statistical parameters such as significant wave height, spectral peak period or mean zero-crossing wave period, spectral shape and directionality. Other parameters of interest can be derived from these.

Wave periods can significantly affect wave drift forces and floating structure motions, therefore a range of wave periods should be examined. For example, when conducting a mooring analysis for the 100 year return period wave, it is advisable to check a set of combinations of significant wave height and spectral peak period selected from a 100 year significant wave height – spectral peak period contour line for the area.

A.7.2.5 Wind statistics

Reference can be made to Reference [78] for guidance on wind loads.

A.7.2.6 Current profile

The most common categories of currents are

— tidal currents (associated with astronomical tides),

— circulation currents (associated with oceanic scale circulation patterns, e.g. loop and eddy currents),

— storm generated currents, and

— soliton currents.

For a given sea state, the total current velocity is the vector sum of the current velocities applicable to the site. In certain geographic areas, current action can govern the design. Consequently, selection of appropriate current velocity profiles requires careful consideration. Reference can be made to References [28] and [78] for guidance on actions due to current.

A.7.2.7 Atmospheric icing

No guidance is offered.

A.7.2.8 Marine growth

No guidance is offered.

A.7.3 Environmental actions on mooring lines

A.7.3.1 General

No guidance is offered.

A.7.3.2 Current-induced actions

No guidance is offered.

A.7.3.3 Ice-induced actions

See ISO 19906 [5].

A.7.3.4 Vortex-induced vibrations of mooring lines

Fluid flow past a slender member can cause unsteady flow patterns due to vortex shedding. At certain critical flow velocities, the vortex shedding frequency can coincide with a multiple or sub-multiple of the natural frequency of vibration of the member resulting in harmonic or sub-harmonic excitations normal to the longitudinal axis of the member, either in-line (parallel to the flow) or transverse (perpendicular to the flow). For mooring lines, the transverse vibrations/excitations can be of concern, as they tend to increase the drag-induced actions.

Four different methods are typically employed to assess the effects of vortex-induced actions on slender cylindrical members:

— simplified assessment of vortex-induced motions and fatigue;

— multi-modal response analysis based on empirical models (and tests);

— computational fluid dynamics methods solving the Navier-Stokes equations;

— laboratory tests.

The method should be chosen according to the specific case investigated. Recognized semi-empirical methods may be applied if the problem characteristics are well within the validity range based on previous relevant experience.

A.7.4 Indirect actions

A.7.4.1 General

No guidance is offered.

A.7.4.2 Frequency ranges

No guidance is offered.

A.7.4.3 Wave-induced actions

The motions of the structure at the wave frequency are an important contribution to the total actions on the mooring system, particularly in shallow water.

Interactions between ocean waves and a floating structure result in motions of the structure that can be conveniently split into three categories:

— first-order motions known as high-frequency or wave-frequency motions;

— second-order motions known as low-frequency motions;

— a steady offset known as mean wave drift.

Wave-frequency motions can be obtained from regular or random wave model test data or computer analysis using either time or frequency-domain techniques, see 19904-1.

Wave-frequency motions have six degrees-of-freedom: surge, sway, heave, roll, pitch and yaw. They are normally considered to be independent of mooring stiffness except for floating systems with natural periods less than 30 s.

However, in some cases, the stiffness of the mooring system and/or risers, etc., can have a significant influence on the wave-frequency motions. One example is a deepwater CALM buoy system where the wave-frequency motions of the buoy are influenced by the stiffness, inertia and drag of the mooring lines, and the corresponding properties of the mid-depth oil offloading lines. In such cases, accurate CALM buoy motions are only obtained by analysis of the entire coupled buoy/mooring/offloading line system. Furthermore, fatigue analysis of the mooring lines and offloading lines should consider this coupled behaviour.

Low-frequency motions are induced by the low-frequency component of the second order wave actions, which, in general, are quite small compared to the first-order actions. Because of this, the low-frequency actions do not play a significant role in the motions in the vertical plane (i.e. roll, pitch and heave motions) where large hydrostatic restoring actions are present. However, in the horizontal plane (i.e. surge, sway and yaw motions), where the only restoring actions present are due to mooring or dynamic positioning systems and production risers, the motions produced by the low-frequency actions can be substantial. This is particularly true at frequencies near the natural frequency of the moored structure. Therefore, in general, only low-frequency surge, sway, and yaw motions are included in a mooring analysis.

Low-frequency motion of a moored structure is narrow-banded in frequency since it is dominated by the resonant response at the natural frequency of the moored structure. The motion amplitude is highly dependent on the stiffness of the mooring system. The motion amplitude is also highly dependent on the system damping so that a good estimate of damping is critical in computing low-frequency motions. There is a substantial degree of uncertainty in the estimation, particularly in damping.

Wave-induced motions of floating structures is discussed in Reference [36].

A.7.4.4 Wind-induced actions

Wind velocity increases with height above the water. If wind speed is given at a reference height other than 10 m it should be adjusted to 10 m using the profile given in ISO 19901-1.

Wind action can be treated as constant or as a combination of a steady component and a time-varying component. The time-varying component is also known as low-frequency wind action. Similar to the low-frequency second order wave actions, low-frequency wind action also induces low-frequency resonant surge, sway and yaw motions. Low-frequency wind actions are normally computed from an empirical wind energy spectrum such as that presented in ISO 19901-1. Low-frequency wind and wave actions are normally combined to yield low-frequency structure motions due to both effects.

Reference should be made to ISO 19904-1 for further information on wind-induced actions.

A.7.4.5 Current-induced actions

Current-induced motions of floating structures are discussed in Reference [36].

For current-induced actions on floating structures susceptible to VIM see A.7.4.7.

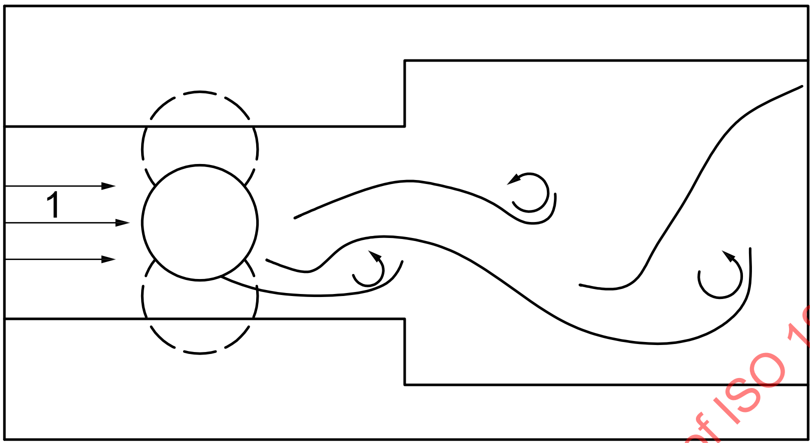
A.7.4.6 Directional distribution

In the evaluation of the action effects on stationkeeping systems, the directional characteristics of the various environmental phenomena should be considered. If all the environmental phenomena come from the same direction, and the moored structure aligns with this direction, then the resulting action effects on the mooring system are usually minimized. However, when waves act at high angles to winds or currents and the moored structure is not aligned with the predominant environmental actions, the resulting action effects are generally higher.

A.7.4.7 Vortex-induced motions of floating structures

A.7.4.7.1 VIM fundamentals

Cylindrical structures exposed to a current create alternating eddies, or vortices, at a regular period. Figure A.7 shows how these eddies appear in the downstream wake of a cylinder.



Key

1 flow

Figure A.7 — Eddies in the downstream wake of a cylinder

The vortex shedding frequency, f_s, is related to the non-dimensional Strouhal number, S, by:

$$f_s = \frac{S V_c}{d} \tag{A.1}$$

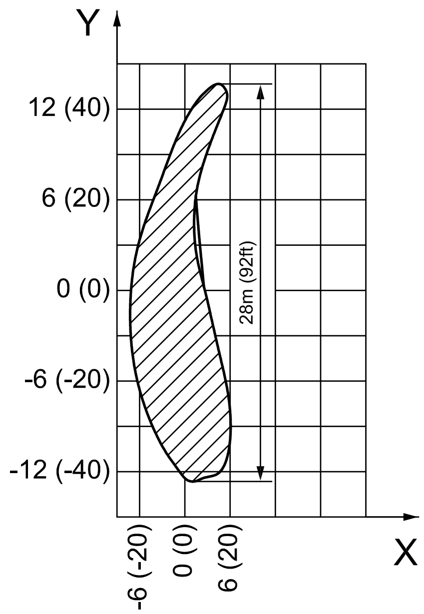
where

V_c is the current velocity, in m/s;

d is the cylinder diameter, in m.

The eddies create alternating lift and drag forces on the cylinder. When one of the cylinder's natural frequencies falls close to the vortex shedding frequency, oscillations of the cylinder can occur. This VIV phenomenon is well known for risers and tendons.

VIV, however, is not restricted to long cylinders. Floating structures made of cylindrical members, such as spars, TLPs and semi-submersibles, experience vortex-induced oscillations when their surge/sway or roll/pitch frequencies are close to the vortex shedding frequency. Figure A.8 shows an example of the motion envelope of a spar subjected to a current of slightly over 1,0 m/s. The period of the motion in the transverse direction in this case is about 180 s, which is close to the natural sway period of the spar. There is also a smaller motion in the in-line direction, characterized by a period of about one-half the transverse period.



Key
X in-line direction
Y transverse direction
Units m (ft)

Figure A.8 — Motion envelope of a spar experiencing VIM

The occurrence of lock-in is related to the non-dimensional reduced velocity:

$$V_r = \frac{V_c T}{d} \quad \text{(A.2)}$$

where

T is the characteristic period, in s;

V_c is the current velocity, in m/s.

The definition of V_r can vary (see A.7.4.7.2.1). If T is the natural period in still water (no current), lock-in can typically occur for values of $4 < V_r < 10$ for transverse VIM. The precise range of lock-in depends on parameters such as the structural shape, vortex mitigation devices, appurtenances, current profile, mass ratio, and damping.

For semi-submersibles and TLPs, the presence of multiple columns leads to a more complicated flow field, which usually also causes yaw excitation in addition to the transverse and in-line responses seen for a single column floating structure such as a spar. In addition, the surge and sway natural periods of a spar and semi-submersible are comparable and, typically, a semi-submersible column diameter is about half a spar hard tank diameter. This has two implications:

a) full lock-in occurs at much lower current velocities for a semi-submersible compared to a spar; this results in VIM being an issue for mooring and riser fatigue rather than mooring strength for a semi-submersible compared to a spar;

b) at higher current speeds, a semi-submersible is generally in the post lock-in regime, i.e., $V_r > 10$. This results in a non-harmonic response with random amplitudes and broader band frequencies, while the in-line and transverse responses are of comparable magnitude.

A.7.4.7.2 VIM design parameters

A.7.4.7.2.1 Reduced velocity

For the six degree-of-freedom motions of multi-member bodies (semi-submersibles, TLPs) the definition of reduced velocity (V_r) is complicated. In general, the definition of V_r involves the eigen-periods of the system under mean load (which depend on the mooring system stiffness and hydrostatic stiffness, and the full 6-by-6 mass and added mass matrices), the characteristic dimension of the body (which can vary with the eigen-mode under consideration), and the characteristic velocity of the incident flow.

The following discussion applies to the VIM of a classic spar transverse to the current direction. In this case, transverse VIM occurs when the vortex shedding period is close to the natural period of the floating structure transverse to the current direction. For a classic spar the relation between VIM response and the natural or observed period of the transverse motion is often given in terms of the reduced velocities $V_{r,n}$ or $V_{r,obs}$,

$$V_{r,n} = \frac{V_c T_n}{d} \text{ and } V_{r,obs} = \frac{V_c T_{obs}}{d} \tag{A.3}$$

where,

V_c is the characteristic current velocity, typically the highest velocity in the current profile;

T_n is the still water natural period of the floating structure transverse to the current direction under mean load;

T_{obs} is the observed period of VIM;

d is the spar diameter.

Note that $V_{r,obs}$ is only defined over the range of current velocities that induce VIM, whereas $V_{r,n}$ is defined for all current velocities. Model test data indicate that VIM is a function of $V_{r,n}$ or $V_{r,obs}$ and VIM is negligible when $V_{r,n}$ is below a threshold value. The range of $V_{r,n}$ or $V_{r,obs}$ where significant VIM can be induced is often referred to as the lock-in range.

$V_{r,n}$ is a function of T_n, which is a function of mooring stiffness and the structure's mass. Mooring stiffness at various offsets can be significantly different, especially for grouped mooring patterns. The structure's mass includes added mass, which is typically determined by analytical tools or model testing. If available, field measurement data should be used to calibrate the added mass values. The transverse stiffness used for calculating $V_{r,n}$ is typically evaluated at the mean offset under current and associated wind and waves. Since the mean offset is dependent on the drag force, which is dependent on the VIM amplitude, the process of selecting the appropriate offset for VIM calculation is iterative. It should be noted that the observed period from model tests or field measurements T_{obs} can be different from the calculated still water natural period, T_n, which is used in most analyses as it is readily available. Calibration of calculated values with available model test or field measurement data is desirable when such data are available.

Since $V_{r,n}$ is a function of current velocity and natural period of the floating structure, VIM can exist under relatively mild currents (for example 0,5 to 1,0 m/s) if the natural period of the vessel is long. This may be the case with deepwater floating structures that have low mooring stiffness relative to their total mass.

A.7.4.7.2.2 Transverse (cross flow) VIM

Transverse VIM occurs when the vortex shedding period is close to the transverse natural period of the floating structure, which in this case typically oscillates in the direction perpendicular to the current in a periodic pattern. Transverse motion is normally expressed as the ratio of single amplitude transverse VIM to column diameter (a/d). However, transverse VIM sometimes has an asymmetric pattern. In this case, transverse a/d should be specified for two opposite directions. Transverse VIM is a function of a large number of parameters such as reduced velocity, floating structure type (spar, semi-submersible, TLP), strake configuration (shape, height and coverage), current characteristics (profile, speed, and direction), and hull appurtenances (anode, chain, fairlead, and pipe), etc.

A.7.4.7.2.3 In-line VIM

In-line VIM is in the direction of the current, and can affect the transverse VIM. In-line motion amplitude is also a function of the parameters discussed above for transverse VIM. In the lock-in range, the in-line motion amplitude is typically much less than the transverse motion amplitude. Field measurement data for a classic spar with an equally spaced spread mooring system indicate in-line motion amplitude of 10 % to 15 % of the transverse motion [155] in lock-in conditions. However, the in-line motion magnitude can be higher if the natural period for the in-line motion is about half of the natural period for the transverse motion (in-line resonance). Also, unsymmetrical mooring system stiffness could result in a VIM trajectory for which the major axis of the VIM is not transverse to the current direction.

Due to a smaller column diameter compared with a spar, multi-column floaters can also respond in the post lock-in region (V_r>10). In this case, the transverse and in-line responses are non-harmonic with random amplitudes and broader band frequencies, and the in-line and transverse responses are of similar magnitudes.

A.7.4.7.2.4 Drag coefficient

Model tests are typically used to determine the drag coefficient, C_d, for use in the design. A base drag coefficient C_{d0} is assumed for the case where a/d = 0.0 (no VIM) and amplification factors are applied to account for VIM effects. The drag augmentation is a function of a/d and V_r and can be expressed as [156], [157], [158]:

$$C_d = C_{d0}\left[1 + k\left({}^{a}\!/_{d}, V_r\right)\right] \tag{A.4}$$

where

k is an unspecified function of a/d and V_r.

The mean drag force on a cylinder is given by

$$F_d = C_d \frac{1}{2} \rho V_c^2 A_p \tag{A.5}$$

where

C_d is the mean drag coefficient (absolute current velocity) in the presence of VIM;

ρ is the density of the fluid;

V_c is the free stream current velocity;

A_p is the projected area.

In the lock-in range, the drag coefficient increases almost linearly with a/d. For a classic spar, where the spar diameter is well defined, the drag coefficient under lock-in condition can be expressed by the following equation:

$$C_d = C_{d0} + f\left(\frac{a}{d}\right) \tag{A.6}$$

where

C_d is the spar drag coefficient with VIM;

C_{d0} is the spar drag coefficient without VIM;

f is a hull specific coefficient.

The coefficient f is hull specific, and is normally determined by model testing. It also depends on the definition of a/d and C_d (extreme or mean a/d, and absolute or relative velocity C_d). Some of the reference publications demonstrate the variability of drag observed in model tests. Such variability can warrant sensitivity checks on drag predictions as part of the mooring design.

A.7.4.7.3 Effects of water depth and current turbulence

While VIM magnitude is generally not a direct function of water depth, VIM and mooring line tension are affected by the change of stiffness in different water depths. Mooring stiffness typically increases with decreasing water depth. The higher mooring stiffness in shallower water can reduce or even suppress VIM under certain conditions, due to a resulting value of V_r less than the lock-in threshold. However, if higher stiffness fails to reduce or suppress VIM, the mooring line can experience a significant increase in line tension. Industry experience indicates that VIM can cause significant line tension increase for typical steel taut leg moorings in water depths of 600 – 1 000 m, where VIM amplitudes can be a significant fraction of the total offsets. The VIM influence on line tension is much smaller in water depths greater than 1 500 m, because mooring stiffness generally decreases, while VIM amplitude remains similar in magnitude regardless of water depth. Although VIM of the same magnitude is likely to be less damaging for deepwater moorings, the VIM effects should still be considered. A significant sea floor slope can result in significantly different anchor depths for different mooring lines, causing directional change of mooring stiffness. This in turn can induce directional VIM response.

While the correlation between the limited available field measurements of spar VIM and model test results does not indicate that turbulence in ocean currents influences spar VIM response, there is evidence from model testing that high levels of turbulence in the model basin can affect VIM response. The structure and intensity of turbulence in ocean currents and the potential impact of current turbulence on VIM remain uncertainties for further observation and investigation.

A.7.4.7.4 Environmental considerations

A.7.4.7.4.1 Current

The most common categories of currents are:

— tidal currents (associated with astronomical tides);

— circulation currents (e.g., the Gulf Stream, the Gulf of Mexico Loop Current and associated eddies, Brazil current);

— storm generated currents;

— internal wave generated soliton currents.

Spar VIM has been detected in the Gulf of Mexico in the presence of eddy currents and hurricane-generated inertial current. Other types of current can also induce VIM.

A.7.4.7.4.2 Environment for strength analysis

Mooring strength analysis under the VIM condition is normally conducted for an extreme current with associated wind and waves. However, the current for the worst-case VIM strength event does not necessarily occur at the 100-year return period current, but could relate to a lower return period current coinciding with VIM lock-in. The metocean criteria should specify current velocity, profile, and direction as well as the intensity and direction (collinear or non-collinear) of wind and wave conditions associated with extreme currents. However, recent experience suggests that consideration should also be given to extreme wind and waves with associated current.

A.7.4.7.4.3 Environment for fatigue analysis

For long-term fatigue analysis, current conditions can be represented by a number of discrete current bins, with each current bin consisting of a reference direction, a reference current velocity and profile, associated wave and wind conditions, and probability of occurrence. Studies also indicate that for some mooring systems, considerable fatigue damage can be caused by a single extreme VIM event, which should also be addressed. The current for the worst-case VIM fatigue event does not necessarily coincide with the 100-year return period current, but could relate to a lower return period. For fatigue analysis of single VIM events, the current criteria should specify the current velocity, profile, direction, and duration (build-up and decay) for current events spanning a range of return periods.

A.8 Mooring analysis

A.8.1 Basic considerations

No guidance is offered.

A.8.2 Floating structure offset

It is recognized that in addition to combining the steady components of wind, wave and current, circumstances can require the addition of VIM to determine the floating structure mean offset. At present, there is no industry consensus on the methodology for combining the mean effects of VIM with the mean effects of wind, wave and current.

A.8.3 Floating structure response

A.8.3.1 Analysis methods

A.8.3.1.1 General

No guidance is offered.

A.8.3.1.2 Frequency-domain approach

No guidance is offered.

A.8.3.1.3 Time-domain approach

The value of a particular response parameter (structure offset, line tension, anchor forces, grounded line length, etc.) realized in a single time-domain simulation will vary about its expected value. Consequently, statistical fitting techniques and repetition of the simulation are required to establish reasonable confidence in the predicted extreme response. The number of repetitions of the simulation that are required will depend upon the extreme value characteristics of the system response parameter and the sophistication of the statistical methods used to predict the maximum value. In particular, the scatter (standard deviation) of realizations of extreme values from individual storm simulations can be expected to increase as the number of low-frequency cycles in the storm duration decreases (as low-frequency natural periods increase).

For turret moored structures, the low-frequency natural period of the structure’s yaw rotation will generally be significantly longer than the surge and sway natural periods. When the yaw natural periods are long, a large scatter (standard deviation) in the realizations of extreme values from individual storm simulations is to be expected. Consequently, a large number of repetitions of the storm simulation are usually required to achieve confidence in the prediction of the maximum response values. Guidance in this respect can be found in [38].

A.8.3.1.4 Combined time-domain and frequency-domain approach

In this approach, the mean and low-frequency responses are simulated in the time domain, which allows for non-linearities in stiffness of mooring lines and risers and in structure actions due to quadratic terms and changes in yaw angle. Constant or variable thruster actions can also be modelled. Transient motions resulting from line breaking or thruster failure can be evaluated by specifying the time of failure in the time-domain analysis. Unlike the full time-domain approach, evaluation of low-frequency damping due to mooring line and riser cannot be included in this simulation because of the absence of wave-frequency components. The damping should be evaluated separately and treated as an input parameter.

Wave-frequency structure motions are calculated separately in the frequency domain from the structure's motion response amplitude operators (RAOs) and the wave spectra. These motions can be combined with the low-frequency motions in two ways. In the first method, the frequency-domain solution of wave-frequency structure motions is transformed into a time history, which is added to the mean and low-frequency structure displacement to arrive at the combined structure displacement. In this case, the wave-frequency time history should be calculated for the same wave train (seed values) used for generating low-frequency time history, and taking into account the instant position and heading of the floating structure (as obtained from the low-frequency time history) so as to yield consistent results.

In the second method, the mean response and the low-frequency response time histories are statistically analysed to determine the extreme values, which are then combined with the extreme values of the wave-frequency response to arrive at the maximum structure offset.

A.8.3.2 Extreme value statistics

It should be noted that the Rayleigh distribution does not always yield conservative predictions of extreme motions. Of particular concern are the passive turret moored structures that will not maintain a constant heading because of low-frequency yaw motions. The extreme response can be significantly affected by variation in structure heading, and using a Rayleigh distribution can substantially underestimate the extreme value.

A.8.3.3 Low-frequency damping

The methodology used to estimate low-frequency viscous damping for the floating structure as a whole is well established, and viscous damping is normally included in the low-frequency motion calculations.

Further details concerning low-frequency wave damping can be found in [40], [42], [43], [44], [45], [46], [67], [69], [70] and [83].

Wave drift damping and mooring system damping, however, are more complex and are often neglected because of a lack of understanding of these damping components. Research indicates that wave drift damping and mooring system damping can be significant. They can even be higher than viscous damping under certain conditions, and neglecting them can lead to significant overestimation of low-frequency motions. In applications where low-frequency motions are an important design factor, such as for large ship-shaped structures, it can be warranted to evaluate damping from all these sources by either an analytical approach or model testing.

Damping is dependent on water depth and the numbers of mooring lines and risers, in addition to the actual sea state and current profile. For permanent moorings, the applied damping should be verified by model tests. A conservative level of damping should be applied in the absence of more accurate information.

A.8.3.4 Riser considerations

No guidance is offered.

A.8.3.5 Vortex-induced motion considerations

A.8.3.5.1 General

A.8.3.5.1.1 Floating structure response

A.8.3.5.1.1.1 VIM response modes

The action induced by vortex shedding on the hull of bluff body structures can cause response in any of the six rigid body modes. Primary concerns for most floating structures are the transverse (sway) response and the in-line (surge) response, which are typically included in a mooring analysis. However, the possible effects of vortex shedding induced actions on other response modes should also be checked. For example, for some floating structures large pitch, roll, or yaw responses or large mean transverse displacements could affect the mooring system.

For multi-column floating structures, in addition to yaw responses, a complex non-harmonic response and/or a non zero-mean transverse response are often observed in model tests. This could be due to the asymmetrical and non-stationary/non-harmonic nature of the fluid flow field which tends be quite complex in the presence of multiple columns and pontoons.

A.8.3.5.1.1.2 VIM response prediction

Model testing is currently the primary tool for VIM predictions because of difficulties in obtaining full-scale response data in a timely fashion, and the lack of a validated numerical or analytical approach. Industry studies suggest, however, that model tests are only able to accurately model certain effects while compromising others. Consequently, confidence in model test results and VIM design criteria should be established through comparison with field measurement data. The reliance on model testing, the limitations of model testing, and limited validation with full-scale data should be recognized as a potential sources of uncertainty in the design process. A more detailed discussion on model testing can be found in A.8.3.5.1.2.

A.8.3.5.1.1.3 Peak value statistics

The design of mooring systems for VIM strength and fatigue are typically based on special criteria developed for VIM extremes. This is a departure from more traditional approaches based on standard deviation and peak value statistics, which are in turn a function of the duration of the extreme environmental event. The traditional approach is not used for VIM because the peak value statistics have not been well established for transverse and in-line VIM, and the duration for the extreme environmental event, for example the 100-year current, is difficult to estimate for many locations.

Preliminary investigation of some full-scale and model test data for the VIM of classic spars in the lock-in range (where the motion is well developed and sustained) indicates the maximum to standard deviation ratio for in-line VIM is about 85 % to 90 % of that determined by a Rayleigh distribution. For transverse VIM in the lock-in range, the ratio of maximum to standard deviation of VIM amplitude can vary from 1,6 to over 2,0 for durations of a few hours to a few days, respectively. These values are given for illustration only, and should not be used for a specific application without further investigation.

A.8.3.5.1.2 Model testing

A.8.3.5.1.2.1 Basic considerations

Model tests are routinely conducted to investigate VIM and VIM mitigation methods. Sound VIM model testing practice should adequately address the following issues:

- geometric scaling;
- dynamic scaling;
- hydrodynamic scaling;

— modelling of appurtenances;

— mooring stiffness characteristics;

— degrees of freedom;

— current direction and profile;

— directional resolution;

— test rig damping;

— blockage (wall) effect;

— length of response record.

Since the early 1990s, significant efforts have been devoted to improve model testing methodology to obtain better predictions of VIM responses. Recent model tests yielded spar VIM predictions that compare reasonably well with field measurements [155]. However, all model tests conducted to date could only accurately model certain parameters while approximating others. Different model testing methodologies and practices can result in different test results. Confidence in model test results and in VIM design criteria should be steadily enhanced through adherence to sound engineering principles and comparison with field measurements where available. The reliance on model testing, the limitations of model testing and limited validation with full-scale data should be recognized as potential sources of uncertainty.

A.8.3.5.1.2.2 Model test parameters

A.8.3.5.1.2.2.1 Flow similitude

Hydrodynamic similitude between prototype scale and model scale fluid flow in the model testing of offshore structures [159] is governed by the Reynolds number and the Froude number.

The non-dimensional Reynolds number, R_e, is defined as

$$R_e = \frac{V_c}{\nu} d \tag{A.7}$$

where

V_c is the characteristic velocity (e.g., flow velocity), in m/s;

d is the characteristic length (e.g., hull diameter), in m;

ν Is the kinematic viscosity of the fluid, in m²/s.

The non-dimensional Froude number, F_n is defined as

$$F_n = \frac{V_c}{\sqrt{g\,d}} \tag{A.8}$$

where, in addition to the symbols for Equation (A.7)

g is the gravitational acceleration in m/s²

Matching the Reynolds and Froude numbers simultaneously for both the model and the prototype (full scale structure) flows, however, is practically impossible. For a model dimension d that is substantially smaller than prototype, either the gravity g needs to be significantly increased, or viscosity v of the testing fluid needs to be significantly decreased. Neither of these changes is practical in a test basin.

For the separated flow regimes that are likely to induce VIM, Reynolds number scaling is the key aspect. Reynolds scaling is particularly difficult to achieve for an offshore floating structure. For spar hull diameters of 20 m – 50 m and current velocities of 1,0 m/s - 2,5 m/s. the Reynolds number for the prototype are in a range of 20 000 000 to 100 000 000. Matching such Reynolds numbers in the model basin would require the model to experience hydrodynamic actions of the same magnitude as that of the prototype, which is obviously impractical. Consequently, two basic testing approaches, supercritical and sub-critical Reynolds number, are used.

a) Supercritical Reynolds number model testing. Testing at supercritical Reynolds numbers is conducted to attain a flow regime similar to the flow experienced in full scale [155], [160], [161]. Supercritical model tests conducted at Reynolds numbers of between 600 000 and 2 000 000 for classic spars have shown good agreement with full-scale (15 000 000<R_e<40 000 000) responses measured in the field. However, supercritical Reynolds number model testing places significant demand on the capacity of the model basin, and to date supercritical model tests have only used a single degree-of-freedom model subject to a uniform current profile.

b) Subcritical Reynolds number model testing. For a cylinder with helical strakes, flow separation in the near field is controlled by the sharp edges of the strakes and not by boundary layer effects [162]. In addition, it is possible to consider the six degrees-of-freedom structure response and include a variable current profile in the model test. Subcritical model tests conducted at Reynolds numbers of between 50 000 and 400 000 for a spar yielded conservative results when compared to full scale (30 000 000<R_e<40 000 000) measurements [155], [163].

A.8.3.5.1.2.2.2 Dynamic similitude

Dynamic similitude generally addresses the structure's rigid body modes of vibration. For the purposes of VIM investigations, the similitude can be limited to modelling only the rigid-body modes that are likely to experience lock-in. For example, a spar can experience lock-in in sway at lower velocities and in roll at higher velocities [164]. In some cases, the two degrees of freedom can actually exhibit coupling (simultaneous lock-in). In such circumstances, it is important that the sway and roll modes and periods be properly scaled. On the other hand, if the transverse sway is the dominant VIM response, then tests with a single degree-of-freedom rigid body mode have shown reasonable agreement between model test and full-scale data [160].

The reduced velocity V_r , introduced in 8.3.5.2, is an important dimensionless parameter for VIM:

$$V_r = \frac{V_c T}{d} \tag{A.9}$$

In this expression, the definition of the characteristic period T can vary. If T is defined as the natural period of the floating structure in still water, VIM lock-in for a classic spar typically occurs for values of $4 \le V_r \le 10$.

In order to achieve proper fluid-structure VIM similarity, the reduced velocity for model flow should match the reduced velocity for the prototype flow. That is, in addition to selections of proper scaling for V_c and d, scaling for period T should also be appropriate.

The mass ratio has a large effect on the range of lock-in, and possibly on the amplitude [165], [166], [167]. The mass ratio for a free floating body is by definition equal to 1,0 (displacement = weight). This mass ratio should be maintained for model tests as well.

A.8.3.5.1.2.2.3 Geometric similitude

A.8.3.5.1.2.2.3.1 General

In order to achieve geometric similitude, the geometric shape of the hull and strakes (if appropriate) for both the prototype and of the model should be accurately scaled. The geometric similitude should extend to any construction openings in the strakes, brackets (which could modify the flow along the strakes), chains, anodes, external pipes and other appurtenances that can affect the flow around the body. Some members, e.g. the truss members of a truss spar, can introduce Reynolds number dependent viscous damping effects. Care should be exercised in modelling these members.

A.8.3.5.1.2.2.3.2 Model scale

For the model to be geometrically similar to the prototype, the shape of the model should be the same as that of the prototype, with a smaller characteristic length. For considerations related to hydrodynamic actions, it is customary to use smaller (1/100 ratio) model scale for high, supercritical Reynolds number model testing and relatively larger (1/50 ratio) model scale for low, sub-critical Reynolds number model testing.

A.8.3.5.1.2.2.3.3 Appurtenances

All details of the hull should be modelled accurately. For spars, this includes all appurtenances such as fairleads, pipes, chains, anodes, risers and flowlines. Details of strakes including cut-outs or holes in strakes should also be modelled correctly. Accurate modelling of appurtenances is particularly important in developing VIM directional sensitivity and in testing effectiveness of VIM suppression devices such as strakes.

For floating structures with rectangular columns, flow separation is less sensitive to the presence of appurtenances.

A.8.3.5.1.2.2.3.4 Model degrees of freedom

Models with single and multiple degrees-of-freedom have been used. For the single degree-of-freedom model, which is mainly used in high (super-critical) Reynolds number testing, only transverse VIM is allowed. For the multiple degrees-of-freedom model, which is mainly used in low (sub-critical) Reynolds number tests, the structure is free to respond in all six degrees-of-freedom. The relative importance of the multiple degrees of freedom model is determined by the level of coupling between motions of different degrees of freedom.

A.8.3.5.1.2.2.3.5 Mooring stiffness characteristics

Two approaches are generally used to model the stiffness distribution of the prototype mooring system. One approach is to use the reduced velocity (V_r) as an independent parameter. In the model tests, the spar response is measured at different reduced velocities. In the design phase, the transverse period of the spar (hence the V_r) is calculated at different offsets. At each offset, the response amplitude used in design is based on the V_r at that location. In this approach, a linear symmetric mooring system can be used for the model test set-up.

In the alternative approach, the actual spread mooring of the spar is modelled. In this case the current speed is the independent parameter. A spar has typically three or four groups of mooring lines. Each mooring line or group of prototype mooring lines is modelled by an equivalent model mooring line. The horizontal force-displacement characteristic of each mooring line or group is modelled by a bi- or tri-linear spring system so as to mimic the non-linear force-displacement characteristic of each mooring line or group. This allows for modelling of the complete non-linearity and asymmetry of the stiffness. For some mooring systems such as the grouped mooring system, the asymmetry can contribute to a highly directional VIM response.

A.8.3.5.1.2.2.3.6 Current direction and profile

VIM response for spars can be sensitive to small changes in current direction. Fine heading resolutions (e.g., at 10º to 15º increments) can be required to capture the maximum spar VIM response. For multi-column floating structures with rectangular or square columns, there is a distinct directional dependency of the VIM response. Typically, no VIM response is observed when the angle of incidence of the current is less than about 15º with respect to the normal to the face of the column.

Tow tests simulate a slab current uniform with depth. In reality, design currents have a profile and current speeds generally decrease with depth. Efforts have been made to simulate shear current profiles in tow, flume and basin tests [168]. Attempts to generate shear current profiles at the model scale generally result in excessive turbulence. Careful consideration should be exercised in interpreting VIM responses in the presence of turbulent flow. Turbulence in laboratory-generated shear flow can be mitigated by using varying density/viscosity stratified liquid layers in the model tests.

A.8.3.5.1.2.2.3.7 Free surface effect

Free surface effects can be important when the Froude number is greater than 0,15. For surface-piercing towing test of a spar hull model, the towing speed is limited by wave resistance (Froude number). High speed towing can result in Froude numbers that far exceeds the full scale Froude number and exaggerates the free surface effects. One way to avoid the excessive wave resistance for high Reynolds number model testing is to tow a completely submerged, horizontally mounted mirror image of double body with a divider plate in the centre. The divider plate is used to prevent flow communication across the divider plate.

A.8.3.5.1.2.2.3.8 Damping

Damping can affect VIM response, therefore the damping (hydrodynamic and mechanical) generated in the model basin should be consistent with the damping expected at full scale. Since mechanical damping can be generated by the testing equipment and is absent in the field, care should be taken to understand the effect of damping on the VIM response and to mitigate such effects [161]. Hydrodynamic damping due to mooring lines and wave effects in the model test should be given careful consideration when estimating the amplitude of full-scale VIM.

A.8.3.5.1.2.2.4 Length of response record

The model response time histories should be sufficiently long to yield meaningful statistics such as standard deviation, significant, and maximum values. The minimum length depends on the periodicity of the VIM response [160]. When the VIM motion is well developed and sustained (e.g., fully locked-in), relatively few cycles are sufficient to establish the maximum VIM amplitude. If the VIM response is modulated (e.g., in the lock-in and lock-out transition regions), longer records should be used to derive meaningful statistical values. While these portions of the records do not produce a large VIM response, they could be important for computing mooring line fatigue. Consequently, sufficient time record lengths should be obtained. The start up transient response should be excluded from the statistical analysis.

A.8.3.5.1.2.3 Current industry practices

A.8.3.5.1.2.3.1 General

As mentioned in A.8.3.5.1.2.2.3.4, two approaches are currently used for spar VIM model tests, focused on testing at either super-critical or sub-critical Reynolds numbers. The former tests are performed using a horizontal, submerged cylinder in a high speed towing tank [155], [160], [161], while the latter are performed using a floating, surface-piercing vertical cylinder with external spring lines simulating the mooring system [163], [164], [168], [169], [170], [171], [172]. The former approach has so far been limited to classic spars.

Model tests are not performed for all spars. VIM response is self-limiting, and for those cases where a bounding analysis indicated that the mooring system is not governed by high current or VIM responses, then VIM tests are not performed [171].

A.8.3.5.1.2.3.2 Super-critical Reynolds number model testing

In this approach, model tests are conducted for model Reynolds numbers in the super-critical range (i.e. R_e>600 000). The basis for testing the hull model at the super-critical R_e regime is the assumption that, once beyond the transition range, model and prototype flow similitude is preserved. Model tests at super-critical Reynolds numbers for classic spar VIM show relatively good agreement with field measurements [160], [161].

High Reynolds number model testing places significant demand on the resources of the model basin and can be performed only at a few test facilities worldwide. An example of the high, supercritical model testing of a classic spar can be found in [160], [161]. The described rig has been used to tow the spar hull model at R_e up to 2 000 000.

A.8.3.5.1.2.3.3 Low Reynolds number model testing

In this approach, the model is either towed at low speed or in-place tested in a flume or wave basin with current generating capability. Froude scaling is not explicitly required. However, the Froude number is typically chosen so that it is less than that of the prototype. The model test Reynolds number is typically in the sub-critical range. Model tests at low Reynolds numbers for a classic spar has shown conservative results compared with field measurements [155]. The conservatism is possibly due to the difference between the current profiles in the model test (uniform) and in the field (non-uniform).

A benefit of this approach is that motions in all six degrees of freedom can be modelled. This allows for responses in the roll and pitch degrees of freedom to be identified and incorporated in the design. It also allows for the hydrodynamic coupling effects between the different degrees of freedom. The ability to use larger models also facilitates more detailed modelling of the hull details and appurtenances. The vertical moored set-up also gives the ability to model the spatial variation (non-linearity and asymmetry) of the prototype mooring system. One additional benefit is that such approach can be carried out in model basins without high-speed tow capability. Examples of the low, sub-critical Reynolds number model testing of spars can be found in [155], [163], [164], [168], [172].

A.8.3.5.1.2.4 Field measurement data compared with model test data

Field measurements of VIM response have been recorded for three classic spars [155], [160], [161]. In the field, the current profiles vary in speed and heading with depth, as opposed to the slab current adopted in the tow tests described earlier. Hence, the model test values should be adjusted to account for such variations.

Of particular interest is one classic spar for which field measurements at super-critical and sub-critical Reynolds numbers are available [155], [161], [163].

A.8.3.5.1.3 Methods to improve mooring design for VIM

A.8.3.5.1.3.1 Polyester rope for middle section

Spiral strand wire ropes are commonly placed at the middle section of mooring lines for spars. The use of polyester ropes in this section can sometimes reduce the line tensions due to VIM because the lower rope stiffness makes the polyester mooring more compliant for large floating structure movements. The use of polyester rope reduces V_r, which in turn can prevent lock-in. Tension variation due to dynamic actions on the floating structure can be lower for polyester mooring. This results in lower fatigue damage to all mooring components including chain, which generally has the lowest fatigue resistance. A sensitivity study investigating the effects of using polyester ropes instead of spiral strands can be found in [170].

A.8.3.5.1.3.2 Spiral strand for top and bottom sections

Chains are commonly placed at the top (structure) and bottom (anchor) sections of mooring lines for spars. The use of spiral strand in these sections can significantly reduce fatigue damage due to VIM because spiral strand has much higher fatigue resistance than chain. This option requires significant hardware modification, which includes replacing the chain jack and the chain fairlead with a linear winch and a bending shoe. The industry has good experience with mooring systems using spiral strand rope, linear winches, and bending shoes.

A.8.3.5.1.3.3 Improved chain fairleads

The chain section in contact with the fairlead is more susceptible to fatigue failure because of the presence of bending forces in addition to tension. Chain fairleads with seven pockets are commonly used for spar moorings. The use of chain fairleads with nine pockets can reduce chain bending, thus reducing chain fatigue damage in this section. In addition, chain fairlead design resulting in a tight fit between the chain and the fairlead pocket can yield a much lower stress concentration factor and longer fatigue life. Alternatively bending shoes that yield low stress concentrations in chain can be used.

A.8.3.5.1.3.4 Strake design

VIM can be reduced by improved strake design. Options include improving strake shape, increasing strake height, and eliminating discontinuities and holes in strakes. To evaluate the effectiveness of these options, a rigorous model test programme should be conducted.

A.8.3.5.1.3.5 Hull appurtenances

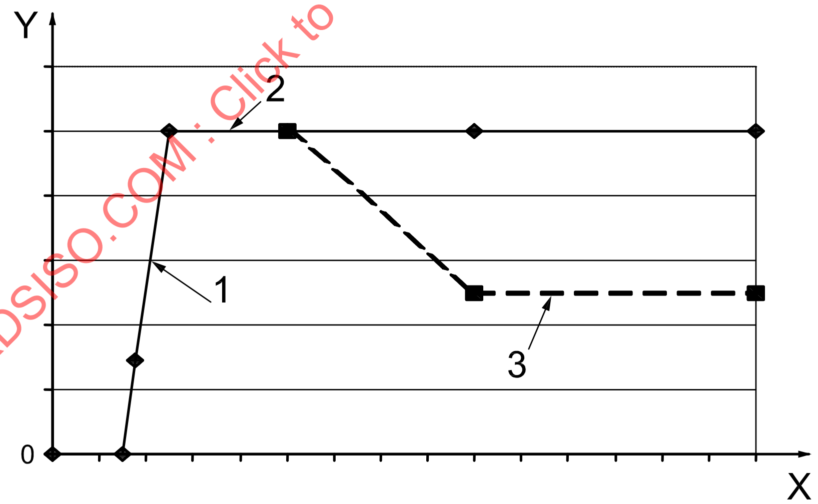
Hull appurtenances such as anodes, chain, fairleads, and pipes can affect spar VIM response. Measures to eliminate or reduce the adverse impact of these appurtenances can reduce VIM. A rigorous model test programme should be conducted to evaluate the effectiveness of these measures.

A.8.3.5.1.3.6 Tightened mooring lines

VIM is not observed in the model basin when V_r is below a threshold value. In some cases, this condition can be achieved in the field by tightening the mooring lines, for example, by using higher initial tensions, or by tightening the mooring system in advance of high current events, thus reducing the natural period of the moored vessel, and eliminating VIM for current speeds below the maximum design value. The adoption of this measure should be based on rigorous model testing and analysis, and on addressing sensitivity to higher current and lower threshold V_r. An operational procedure to ensure a tight mooring during high current events should also be developed [173] and included in the marine operations manual.

A.8.3.5.2 Criteria for VIM strength analysis

An example of a design curve plotting V_r vs. a/d is presented in Figure A.9 showing the locked-in transition, locked-in region, and the locked-out region. This type of curve is typically used to define the VIM response amplitude. For most spars and other moored floating structures experiencing VIM, the response amplitude varies with current direction for the same reduced velocity. Particular attention should be applied to defining the VIM response vs. current heading when setting the design criteria.



Key

X reduced velocity (V_r)

Y VIM amplitude (a/d)

1 locked-in transition

2 locked-in

3 possible locked-out

Figure A.9 — Example VIM amplitude vs. reduced velocity

A.8.3.5.3 VIM strength analysis method

Mooring analysis for high current VIM conditions can require special computer software that is capable of modelling VIM in the frequency domain or time domain. The following simplified analysis procedure can be used if the associated waves and winds effects can be ignored. An example of such a strength analysis can be found in Reference [170].

1) Select a current direction.

2) Determine the mean vessel offset under the design current with associated wind and waves based on an estimated C_d. To yield realistic results, spar set-down should be considered.

3) Calculate in-line and transverse VIM and C_d based on the design criteria established according to A.7.4.7.2.4. If this C_d value is significantly different from the estimated C_d in Step 2, iterations can be required.

4) Determine the envelope of possible maximum vessel offsets including the effects of current/wind/wave vessel offsets (Step 2), and in-line and transverse VIM (Step 3).

5) Determine line tensions and anchor loads corresponding to the envelope of possible maximum vessel offsets calculated in Step 4 by static mooring analysis.

6) Evaluate additional line tensions and anchor forces due to line dynamics, which are superimposed on the quasi-static values obtained in Step 5.

7) Repeat Steps 1 – 6 to obtain line tensions and anchor forces for other current directions.

8) Identify the worst direction for design check.

A proper analysis procedure should include the effects of wave and wind induced motions in combination with the current induced offsets and VIM.

A.8.3.5.4 Basic considerations for VIM fatigue analysis

No further guidance is provided.

A.8.3.5.5 VIM fatigue analysis for long-term and single extreme event

The reason for recommending that a fatigue analysis of the 100-year VIM event should be considered is that, in the long-term fatigue analysis, a single extreme VIM event is spread over the range of current directions and so the fatigue damage is spread over different lines. However, the 100-year loop current event typically does not appear in the long-term fatigue analysis as a single bin lasting 40 days with an almost constant current direction and slowly varying current speed. Although the single 100-year VIM event has a low probability of occurrence, it is important for the designer and operator to know if fatigue failure in a single extreme current event that is nearly constant in direction is likely.

The recommended procedure for long-term fatigue damage evaluation is as follows. An example of fatigue analysis can be found in Reference [170].

1) The long-term current events can be represented by a number of discrete current bins. Each current bin consists of a reference direction and a reference current velocity with associated wave and wind conditions. The probability of occurrence of each current bin should be specified. The number of reference directions depends on the directionality of the current at the site, and the specified directions should include those for which significant VIM is predicted. The minimum number of reference current velocities normally falls in a range of 10 to 50. Fatigue damage prediction can be fairly sensitive to this number for certain mooring systems, and therefore it is best determined by a sensitivity study.

2) Select a current bin and calculate the duration t_i for the current bin in a year based on the probability of occurrence for that combination of current velocity and direction.

3) Determine the natural period T_n of the moored spar under the current bin without VIM based on an estimated C_d.

4) Specify extreme in-line and transverse a/d values for the current bin based on available model test or field measurement data. The mean a/d for fatigue analysis can be evaluated by multiplying the extreme a/d by a coefficient g, which should be determined by available model test or field measurement data.

5) Determine in-line and transverse VIM amplitude coefficient C_v, which is a function of reduced velocity, and is equal to 1,0 at peak VIM under locked-in conditions.

6) Calculate the reduced velocity for the current bin and further modify the mean in-line and transverse a/d (Step 4) by C_v.

7) Determine drag coefficient C_d for the current bin based on the modified mean transverse a/d (Step 6). If this C_d value is significantly different from the estimated C_d in Step 3, iteration can be required.

8) Perform VIM mooring analysis based on the modified mean in-line and transverse a/d (Step 6), and C_d (Step 7), using the procedure for strength design. Determine the average tension ranges R_i, and the corresponding average response period T_i from the time trace of line tensions for a few VIM cycles. The average response period T_i can vary due to the relative orientation of the mooring line and current.

9) Determine number of cycles to failure N_i corresponding to R_i for the mooring component of interest using an appropriate T-N equation. Chain usually has the shortest fatigue life, and chain fatigue life at fairleads is further reduced because of additional stress concentration from bending. Stress concentration factor accounting for bending at fairleads should be determined by testing or by finite element analysis. A factor f_c, which is defined as the ratio of chain stress concentration factor at the fairlead to that away from the fairlead, can be used for calculating fatigue life of chain links at the fairlead. The factor f_c can vary significantly depending on the number of fairlead pockets and the fit between the chain and the fairlead. This factor can be as low as 1,2 for a seven-pocket tight fit fairlead, but it can be higher for a loose fit fairlead. The value of N_i is reduced by a factor of f_c^m at the fairlead, where m is the inverse slope of the T-N equation.

10) Calculate the annual fatigue damage for the i-th current bin:

$$D_i = \frac{\left(t_i / T_i \right)}{N_i} \tag{A.10}$$

11) Repeat Steps 2 to 10 for other current bins.

12) Determine cumulative fatigue damage for all current bins, which is combined with the fatigue damage from wind and waves to obtain total fatigue damage D_i (see 9.3.3.3 for methods to combine fatigue damage). The predicted fatigue life is $1/D_i$ (years), which should be greater than the service life times a factor of safety.

A.8.4 Mooring line response

A.8.4.1 General

Permanent mooring systems should be designed for two primary considerations: extreme line tension values and fatigue. Therefore, analysis for extreme response and fatigue damage should be performed. For mobile moorings, only the analysis of extreme response is required.

The analysis procedure described in this subclause can be applied directly to spread mooring systems, as well as internal and external turret mooring systems. For systems where the mooring is connected to the structure through a buoy (CALM system) or through a riser (turret-riser system), a similar analysis procedure will apply. However, evaluation of wave actions on the buoy or riser and transformation of the structure motions to the chain table through the buoy or riser require special consideration. Model testing or analysis with specialised tools is often required. These analyses are not covered in this part of ISO 19901.

For a CALM system with hawsers, guidelines given in [86] can be used for model testing, design and analysis of the hawsers. The basis for the mooring analysis procedure can be found in [58] and [59].

Extreme responses normally govern the design of the FPS mooring. They include structure offset, mooring line tension, anchor forces, and suspended line length. The environmental conditions for extreme response are described in 6.4.

A.8.4.2 Quasi-static analysis

The quasi-static method is not recommended for the final design of a permanent mooring. However, because of its simplicity, this method can be used for temporary moorings and preliminary studies of permanent moorings with higher safety factors.

A.8.4.3 Dynamic analysis

Several dynamic analysis techniques are available. The distinguishing feature among these techniques is the degree to which non-linearities are treated. There are four primary non-linear effects that can have an important influence on mooring line behaviour.

a) Non-linear stretching behaviour of the line

The strain or tangential stretch of the line is a function of the tension magnitude. Non-linear behaviour of this type typically occurs only in synthetic fibre rope mooring lines. Chain and wire rope can be regarded as linear. In many cases the non-linearity can be ignored and a linearized behaviour assumed using a representative tangent or secant modulus.

b) Changes in geometry

The geometric non-linearity is associated with large changes in shape of the mooring line.

c) Fluid loading

The Morison equation is most frequently used to represent fluid loading effects on mooring lines. The drag load on the line is proportional to the square of the relative velocity (between the fluid and the line) and is hence non-linear.

d) Bottom effects

In most mooring designs, a considerable portion of the line is in contact with the sea floor. The interaction between the line and the sea floor is usually considered to be a frictional process and is hence non-linear. In addition, the length of grounded line constantly changes, causing an interaction between this non-linearity and the geometric non-linearity.

Two methods, frequency-domain analysis and time-domain analysis, are commonly used for predicting dynamic mooring forces.

In the time-domain method, all of the non-linear effects can be modelled. The elastic stretch is mathematically modelled, the full Morison equation is included, the position of the mooring line is updated at each time step, and the bottom interaction is included using a frictional model. The general analysis implies the recalculation of each mass term, damping term, stiffness term, and action at each time step. Hence, the computation can become complex and time consuming.

The frequency-domain method, on the other hand, is always linear, based on the principle of linear superposition. Hence, all non-linearities should be eliminated, either by direct linearization or by an iterative linearization approach, as listed here.

a) **Line stretching**

The line stretching relationship should be linearized and a definite value of the modulus assumed at each point. The modulus cannot be a function of line tension but can vary along the line. This is usually an acceptable assumption even in the case of synthetic mooring lines and, in most cases, a suitable linearization can be achieved.

b) **Geometry change**

In the frequency-domain method it is assumed that the dynamic displacements are small perturbations about a static position. The static shape is fixed and all geometric quantities are computed based on this position. The mass, added mass, stiffness, etc. are computed only once. Changes in catenary shape due to the dynamic motion contribution are generally not severe. Hence, a linearization about the mean position is generally acceptable.

c) **Fluid loads**

The non-linear term in the Morison equation should be linearized by replacing the quadratic relative velocity relationship by an equivalent linear relationship. The linearization should take into account the frequency content of the line motion spectrum.

d) **Bottom effects**

The frictional behaviour between the grounded line and the sea floor cannot be represented exactly in the frequency domain. Only the average or equivalent behaviour of the line can be postulated and included. This simplification should be adjusted to the design objective, i.e. different models are generally required for the fatigue and the extreme tension evaluations.

The relative influence of various non-linearities is a function of numerous parameters, particularly water depth, line composition and motion magnitude. Methods to approximate non-linearities in the frequency domain should reflect the importance of the various parameters.

A.8.5 Line tension

No guidance is offered.

A.8.6 Line length and geometry constraints

No guidance is offered.

A.8.7 Anchor forces

No guidance is offered.

A.8.8 Typical mooring configuration analysis and assessment

A.8.8.1 Frequency-domain analysis for spread mooring systems

The following procedure is recommended.

a) Determine the environmental conditions such as wind and current velocities, significant wave heights and representative wave periods, their relative directions, storm duration and wind and wave spectra for the limit state of interest.

b) Determine the mooring pattern, the characteristics of the mooring line segments to be deployed, and the initial pre-tension.

c) Determine the structure’s wind action and current action coefficients, and develop the hydrodynamic model of the system including structure, riser and mooring.

d) Determine the mean environmental actions acting on the hull.

e) Determine the structure's mean offset due to the mean environmental actions using a static mooring analysis approach, including elastic line stretch and friction.

f) Determine the structure's low-frequency motions. Since calculation of low-frequency motions requires knowledge of mooring stiffness, use the mooring stiffness at the mean offset determined in e).

g) Determine the significant and maximum wave-frequency structure motions using an appropriate motion analysis tool.

h) Determine the extreme values of the structure's offset, S_{extreme}, and the corresponding suspended line length, quasi-static tension, and anchor load using the static mooring analysis tool.

i) If only a quasi-static solution is required, skip this step; otherwise determine the most probable maximum line tension and most probable maximum anchor force using a frequency-domain or time-domain dynamic mooring line analysis tool, see 8.3.5, 8.5.2 and 8.8.

j) Compare the extreme value of the structure offset and suspended line length from step h) and extreme line tension values and anchor force from step h) or step i) with the design criteria in Clause 10. If the criteria are not met, modify the mooring design and repeat the analysis.

A.8.8.2 Frequency-domain analysis for single point mooring systems

The following procedure is recommended.

a) Determine the environmental conditions such as wind and current velocities, significant wave heights and representative wave periods, their relative directions, storm duration, and wind and wave spectra for the limit state of interest.

b) Determine the mooring pattern, the characteristics of the mooring line segments to be deployed, and the initial pre-tension.

c) Determine the structure’s wind action and current action coefficients, and develop the hydrodynamic model of the system including structure, riser and mooring.

d) Calculate the combined mean environmental yaw moment about the mooring point due to wave, wind, and current as a function of structure heading. These yaw moments may be evaluated from model tests or from calculated wind, current and wave drift actions.

e) From the mean environmental yaw moment, determine equilibrium headings and their stability. Stable equilibrium headings occur where the total environmental yaw moment is zero and a perturbation of the structure heading results in a yaw moment opposed to the direction of the perturbation.

f) Determine the yaw rotational stiffness at the equilibrium heading. For an unrestricted mooring point (unlocked turret) the yaw rotational stiffness is the rate of change of the mean environmental yaw moment with respect to a change in heading.

g) Determine the standard deviation of the structure’s low-frequency yaw response about the stable equilibrium headings using a motion analysis tool. This requires knowledge of the low-frequency yaw moment spectrum, the structure's yaw moment of inertia and added moment of inertia about the mooring point, the yaw rotational stiffness, and the structure and mooring system yaw damping. All of the above should be determined for the stable mean structure heading under consideration.

In the absence of better information, the linearized yaw damping coefficient about the mooring point can be estimated from the sway damping as follows:

$$C_{Rz} = \frac{1}{3} C_y \frac{\left(l_1^3 + l_2^3\right)}{\left(l_1 + l_2\right)} \tag{A.11}$$

where

C_{Rz} is the linear yaw damping coefficient, newton metres per radian per second [N m/(rad/s)];

C_y is the linear sway damping, in newtons per metre per second [N/(m/s)];

l_1 is the length of structure forward of the mooring point, in metres (m);

l_2 is the length of structure aft of the mooring point, in metres (m).

h) Calculate three design headings, taking into account both the mean equilibrium heading and the yaw motions, see below.

i) For each of the three design headings, follow the procedure for spread mooring analysis described in A.8.8.1 to calculate mooring system response.

The design headings at which the mooring system response is calculated may be taken as the stable equilibrium headings of the structure under mean environmental actions, and mean plus or minus one standard deviation of the low-frequency yaw motion.

A.8.8.3 Time-domain analysis

The following procedure is recommended.

a) Determine the environmental conditions such as wind and current velocities, significant wave heights and representative wave periods, their relative directions, storm duration, and wind and wave spectra for the limit state of interest.

b) Determine the mooring pattern, the characteristics of the mooring line segments to be deployed and the pre-tension.

c) Determine the structure's wind action and current action coefficients, and develop the hydrodynamic model of the system including structure, riser and mooring.

d) Perform a time-domain simulation for the storm duration using a time-domain mooring analysis tool. Repeat the simulation many times using different wave and wind time histories derived from the input spectra, see 8.3.1.3.

e) Use statistical analysis techniques to establish the maximum values for structure offset, line tension, anchor forces, and line geometry parameters.

f) Compare the results from step e) with the design criteria in Clause 10 and with the geometry constraints described in 8.6.

This procedure only refers to a fully coupled analysis of the structure and its mooring. Partially-coupled analyses can yield reasonable approximations but require special attention in their implementation in order to obtain realistic results.

A.8.9 Thruster-assisted moorings

A.8.9.1 General

Thruster-assisted mooring (TAM) systems should be designed so that, as far as is reasonably possible, there are no common single point failures. A series of sea trials should be planned in order to verify the thruster system FMEA and, as far as is reasonably practicable, to demonstrate the effects of the various failure modes and ensure that both equipment and procedures are in place to safely deal with failures. Typical failure modes for thruster systems are

— blackout — even on the most advanced units there is a risk, albeit small, of total loss of all electrical power,

— partial blackout — loss of one main switchboard or engine room,

— one thruster giving full power in an unwanted direction for 30 s to 40 s before it is stopped,

— one thruster failing,

— one gyro compass giving an incorrect heading that becomes increasingly incorrect with time until the operator takes action,

— one position reference giving increasingly wrong data that are initially accepted by the operator or the control system,

— one tension meter, pay-out meter or similar device failing or giving incorrect readings (either too high or too low),

— total failure of one unified propulsion system, or

— total failure of one automatic control system.

The criticality of failures should be assessed

a) when all equipment is available and functioning as expected, and

b) in various degraded conditions.

The principal results of the thruster system FMEA and availability analysis will be the definition of the worst single failure. When mean times to repair are long or component and subsystem reliability is low, the definition of the worst single failure should allow for system availability. This is particularly important for structures that are to remain on location permanently, as a calm weather window is generally required for certain maintenance and repair operations. For example, if the repair or replacement of a broken thruster requires calm weather, then the probability that a second failure occurs before the broken thruster is repaired or replaced should be considered. A mechanical and electrical systems availability analysis, in which the mean time to repair is conditional on site-specific weather criteria, may be used to evaluate thrust availability over the design service life of the installation.

Thrusters can be used to assist the mooring system by reducing the mean environmental actions, controlling the structure's heading, damping low-frequency motions, or a combination of these functions. Semi-submersibles generally have azimuthing thrusters, whereas ship-shaped structures usually have tunnel or azimuthing thrusters, and both can have main propellers. Generally, semi-submersibles have greater symmetry of environmental actions and effective thrust than ship-shaped structures. Permanent installations, such as FPSs, which are generally not dry-docked on a regular basis, can have lower thruster availability, particularly in winter months, because of difficulties in demounting and repairing thrusters.

In order to provide practical guidance, it is necessary to quantify the allowable thrust used in performing mooring analyses for intact, redundancy check and transient conditions. In determining the allowable thrust the following issues should be taken into account:

— the efficiency of the thrusters and losses due to structure motions, current, thruster/hull and thruster/thruster interference effects, and any directional restrictions;

— the probability of partial or total loss of thrust. FMEA and system availability analyses should be performed to identify the worst system failure to be considered concurrent with the design situation;

— the efficiency of the thruster control system and operators in achieving optimum use of the thrusters. This will depend upon the type of thruster control system and its mode of operation.

It is recommended that the allowable thrust used in the mooring analysis is verified during thruster-assisted system sea trials.

A.8.9.2 Analysis conditions

A system dynamic analysis is normally performed using a three axis (surge, sway and yaw) time-domain simulator. This simulator generates the mean offset and low-frequency structure motions and thruster responses corresponding to specific environmental action time records. In this analysis, constant wind, current, steady wave drift actions, and the low-frequency wind and wave drift actions are typically included. Wave-frequency wave actions, which are not countered by the thruster system, can be excluded in the simulation. The wave-frequency motions are computed separately using a structure motion program and added to the output from the time-domain simulator. To obtain proper extreme values from the time-domain simulation, it is usually necessary to generate a number of action and response records for the storm duration and calculate extreme values using a statistical approach.

A.8.9.3 Determination of allowable thrust

A.8.9.3.1 General

The following subclauses provide guidelines for the determination of the thrust generated by various types of propulsion device. Also addressed is the influence of the installation and arrangement of the propulsion devices, which often leads to a reduction of the available effective thrust (net force acting upon the structure).

The guidelines apply to typical propulsion devices and installation scenarios for DP or TAM controlled structures supporting offshore operations. These include the following:

— open and nozzled propellers installed in the stern of a ship-shaped structure (conventional main propulsion arrangement);

— azimuthing or fixed direction, nozzled thrusters installed under the bottom of a hull;

— tunnel thrusters installed in a transverse tunnel in a hull.

Two methods of thrust evaluation are provided.

a) Tables and figures for quick and rough estimates that can be used for the design of TAM and preliminary design of a DP system.

b) References for more rigorous determination of available effective thrust, which can be used for the final design of a DP or TAM system.

The estimated available effective thrust as determined herein should be further reduced under certain conditions as specified in 8.9.3. Much of the work on allowable thrust is based on References [71], while [63] gives detailed background information on propeller design and allowable thrust.

A.8.9.3.2 Performance assessment

The performance of a conventional propeller, designed to power a structure at a certain speed, is normally expressed by the efficiency of the propeller. During stationkeeping, however, the propeller operates at zero inflow velocity (or at very low speeds), and the application of an efficiency expression is not feasible. A popular expression for the performance of a propeller in stationkeeping application is the specific thrust: propeller thrust per horsepower.

Every propeller designed for stationkeeping delivers maximum thrust at zero inflow velocity. Even in the case of a constant power operation (which is feasible, for example, with controllable pitch propellers, or fixed pitch propellers driven by certain prime movers), the propeller thrust decreases with increasing inflow velocity. Inflow velocity is caused by current speed, movement of the structure, or the jet from another propulsion device. For the analysis of the stationkeeping propeller, the maximum thrust at zero inflow (or bollard pull condition) will be considered the benchmark performance.

To determine the available effective thrust (or net action acting upon the structure), the propeller thrust at zero inflow velocity should be calculated first. This thrust should be corrected by applying thrust deduction factors. These factors depend on the following:

— propeller/thruster installation geometry and arrangement;

— inflow velocity into the propeller;

— propeller sense of rotation (ahead or reverse operation).

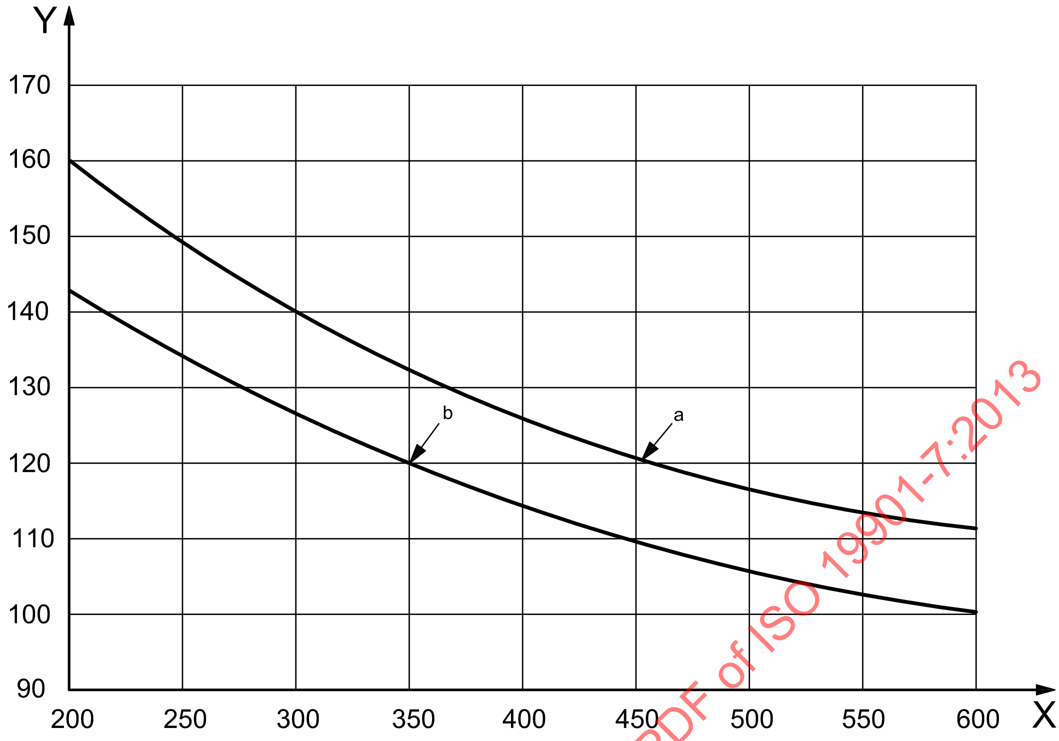
A.8.9.3.3 Propeller thrust at zero inflow velocity

A.8.9.3.3.1 Open propellers

Figure A.10 can be used for quick determination of the propeller thrust at zero speed for an open propeller. Required input data are propeller diameter and the power applied. The diagram clearly indicates that, for a given power, the thrust increases with increasing propeller diameter. It also indicates that for a given propeller the specific thrust increases with decreasing power per unit propeller area. Detailed information and data for the design and performance calculation of open propellers is provided in References [39], [77], [81], and [87].

A.8.9.3.3.2 Nozzled propellers

Figure A.11 allows quick determination of propeller thrust at zero speed for a nozzled propeller. The same basic considerations apply as for open propellers. A comparison between Figure A.10 and A.11 also indicates the considerable increase in thrust available to a nozzled propeller in comparison with an open propeller of the same diameter and power load. Detailed information and data for the design and performance calculation of nozzled propellers is provided in References [80] and [88].



Key

X power/propeller disk area, kW/m^2

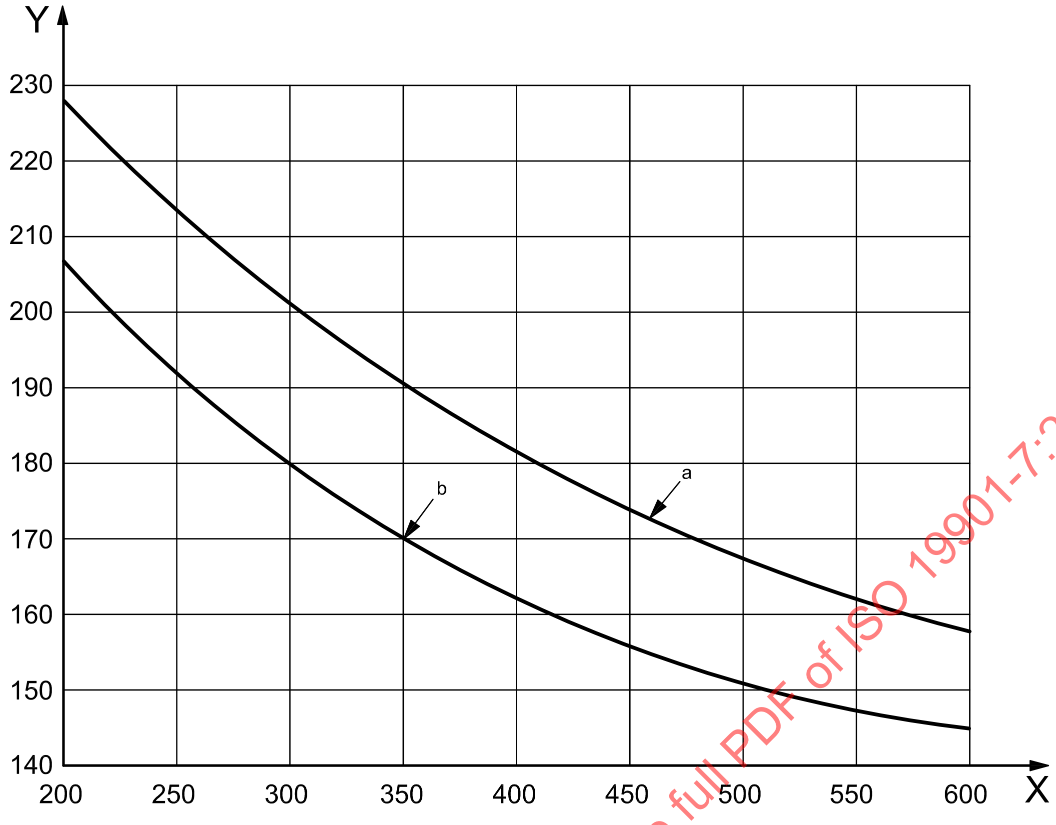
Y specific thrust, N/kW

NOTE Power delivered to the propeller per unit propeller area.

[a] Optimum value.

[b] Low value.

Figure A.10 — Propeller thrust – open propellers



Key

X power/propeller disk area, kW/m^2

Y specific thrust, N/kW

NOTE Power delivered to the propeller per unit propeller area.

[a] Optimum value.

[b] Low value.

Figure A.11 — Propeller thrust – propellers with nozzles

A.8.9.3.4 Calculation of thrust deductions

A.8.9.3.4.1 Propellers installed at the stern of a ship-shaped structure

Propeller suction creates a low pressure field upstream of the propeller, resulting in a reduction of the available propeller thrust. At zero inflow velocity, and during ahead operation, this reduction amounts to approximately 5 % of the propeller thrust. During astern operation, the reduction is about 15 % to 20 %. Detailed data regarding the propeller/hull interaction for conventional vessels are included in References [39], [87], and [81].

A.8.9.3.4.2 Right angle gear thruster propellers

The presence of the gear housing and support struts in the flow to the propeller causes a reduction in thrust. For a thruster of average design, this deduction is about 10 %. In cases where the diameter ratio of gear housing to propeller exceeds 0,45, a deduction of 15 % may be applied.

A.8.9.3.4.3 Thrust deduction due to inflow velocity

For a propeller applied for stationkeeping, propeller operation in certain inflow velocities is caused by currents as well as by the wake created by thrusters operated in the vicinity. Table A.1 indicates approximate deductions of thrust as a function of the inflow velocity. An accurate prediction for the performance of ducted or open propellers at certain inflow velocities is feasible by a detailed analysis, see References [39], [77], [80], [81], [87] and [88]. Information regarding the thrust losses caused by the mutual interference of thrusters can be found in References [61] and [62].

Table A.1 — Correction factor for inflow velocity

Propeller type	Inflow velocity m/s			
	0,5	**1,0**	**1,5**	**2,0**
Open propeller	0,951	0,903	0,854	0,806
Nozzled propeller	0,942	0,883	0,825	0,767

A.8.9.3.4.4 Thrust deduction due to oblique inflow cross-coupling effects

The operation of a propeller in an inflow other than parallel to the propeller axis alters the performance characteristic. Deductions due to inflow velocity can be reduced. However, the creation of cross-coupling actions can cause deduction from the overall balance of actions. The directions of these actions are orthogonal to the propeller axis. These effects are the least researched subjects in propulsion for dynamic positioning. Information and qualitative data can be found in References [26], [62], [84], [85].

A.8.9.3.4.5 Thrust in reverse operation

Some of the thrust producing devices applied for dynamic positioning need to reverse the operation of the propeller to produce thrust in reverse direction. Azimuthing thrusters typically produce thrust in one direction only. They control the direction of thrust by controlling the azimuth angle.

Some thrusters, such as tunnel thrusters or fixed direction nozzled thrusters, are designed as bi-directional devices and are capable of generating approximately equal amounts of thrust in both directions. Propellers optimized for operation in one direction (the majority of marine propellers) are subject to severe deductions while operating in reverse mode. Table A.2 indicates values for thrust losses of nozzled propellers from Reference [62].

Table A.2 — Thrust losses in reverse condition

Nozzle type	Loss %
Symmetric nozzle	5 to 10
Non-symmetric nozzle, elliptic blades	10 to 25
Non-symmetric nozzle, cambered blades	25 to 50

A.8.9.3.4.6 Thrust deduction due to propeller/hull interaction

a) Coanda effect

The high velocity wake from a propulsion device installed under the bottom of a structure can cause areas of low pressure at the hull that result in considerable deductions from the available thrust actions. The magnitude of these deductions depends on the location of the propeller relative to the centreline of the hull, the distance of the propeller from the hull, the radius of the bilge, and the draft of the structure. A correction factor from 5 % to 15 % should be applied to account for this hull interaction. Sources of information and data regarding the thrust losses due to propeller/hull interference are included in [26] and [62].

b) Twin-hull interaction

This effect occurs with twin-hull semi-submersibles having rotatable, under-the-hull-mounted propulsion devices. At certain azimuthing angles, the propeller jet from the thruster is directed towards the neighbour hull, causing a resistance opposite to the direction of the thrust. This effect can be amplified by the above mentioned Coanda effect. Little information regarding these effects is available. The magnitude of the thrust losses depends on the thruster installation geometry and the configuration of the semi-submersible hulls. Countermeasures (which apply also to the Coanda effect) include horizontally tilting the propeller axis downwards or fitting guide vanes to the exit of the nozzle. Both methods deflect the propeller jet away from the neighbour hull. An indication has been found of an average thrust loss of 10 % to 15 % due to the above discussed phenomena, with peak losses of over 50 % at some positions and in particularly unfavourable conditions [18].

A.8.9.3.5 Performance of tunnel thrusters

A.8.9.3.5.1 General

Despite some similarities, tunnel thrusters differ in many ways from the other propulsion devices. They are analytically treated as axial flow pumps. As with marine propellers, the thrust increases with decreased power per unit propeller area. A large propeller diameter yields a high thrust at a given power. The tunnel thruster is subjected to thrust deductions by factors typical for axial flow pumps; the major contributors to the reduction in the net thrust output are restrictions in the flow to and from the impeller, as well as tunnel entrance and exit losses.

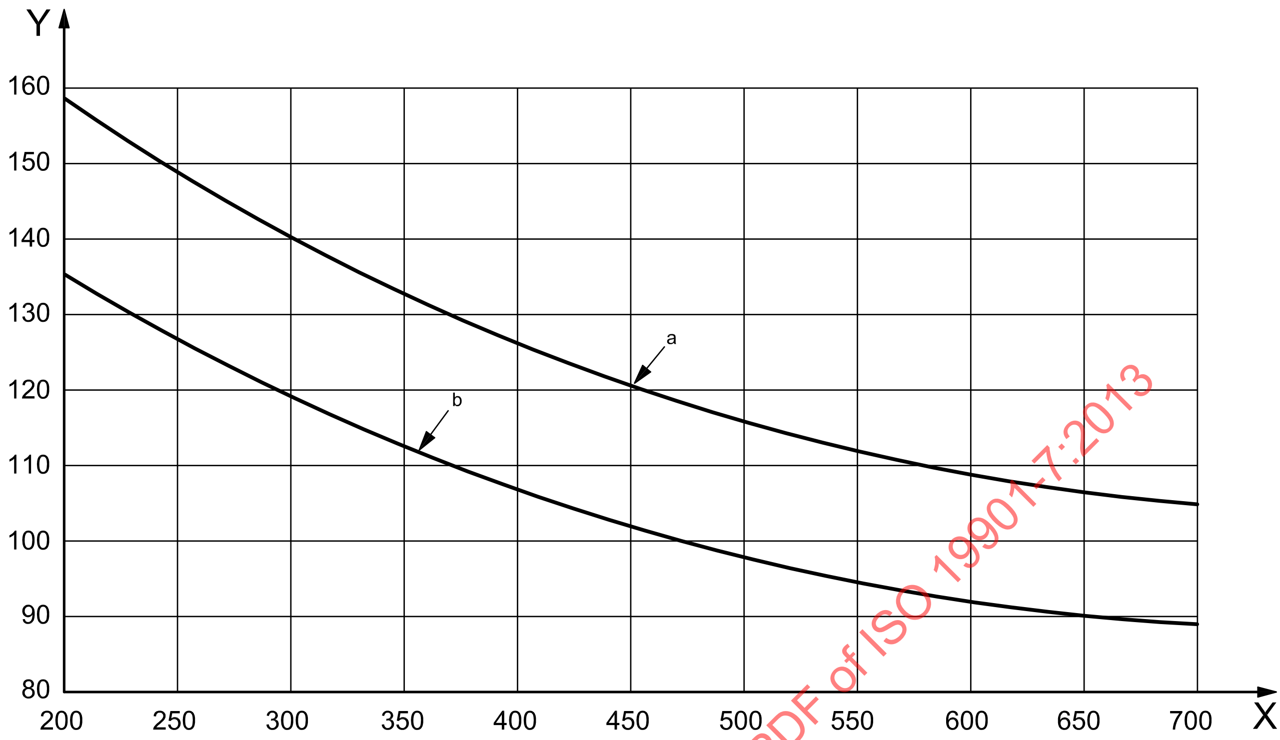
A.8.9.3.5.2 Side force of tunnel thrusters

Figure A.12 can be used for quick determination of the side action of a tunnel thruster, and assumes optimum installation geometry. The tunnel length is about twice the propeller diameter. The hull is perpendicular at the tunnel exits. The exits are conically shaped. No protective bars restrict the tunnel ends. The impeller/hull interaction losses are included.

A.8.9.3.5.3 Thrust deductions for tunnel thrusters

In addition to the thrust losses due to the installation geometry typically associated with tunnel thrusters, further thrust losses can occur during certain operational conditions. The performance prediction of a tunnel thruster is based on a nominal design submergence of the tunnel. If this submergence is decreased due to a reduction in the draft, or due to motions of the structure, the thruster impeller will ventilate (sucking air) and/or cavitate. Both cause a reduction in impeller thrust.

The analytical determination of the losses due to the motions of the structure is complex. First, a relative motion analysis should be performed for the environmental conditions in which the structure is expected to operate. With these data, i.e. periodic variations of the submergence at the tunnel location, the thrust losses during the operation of the impeller at reduced submergence can be calculated [62], [79].



Key

X power/propeller disk area, kW/m^2

Y specific thrust, N/kW

NOTE Power delivered to the propeller per unit area.

a Optimum value.

b Low value.

Figure A.12 — Side force — Tunnel thrusters

A.8.9.4 Load sharing

No guidance is offered.

A.8.10 Transient analysis of floating structure motions

Transient analysis can govern when mean actions and offsets dominate the floating structure response.

A.9 Fatigue analysis

A.9.1 Basic considerations

A valid alternative S-N approach for the determination of fatigue life in wire rope, chain, connecting links and synthetic fibre rope is presented in DNV POSMOOR [29] and summarized in the following subclauses. Except where explicitly stated here, the provisions of Clause 9 equally apply to the S-N approach.

The nominal stress ranges, S (in megapascals), are computed by dividing the corresponding tension ranges by the nominal cross-sectional area of the component, in square metres, i.e.

$$\frac{2\pi d^2}{4} \quad \text{for chain} \tag{A.12}$$

$$\frac{\pi d^2}{4} \quad \text{for steel wire rope} \tag{A.13}$$

where d is the component diameter, in metres (m). For chain, d is the diameter of the bar forming the chain link; for wire rope, it is the outside diameter of the wire.

The relationship between the stress range (double amplitude), S, and the number of permissible cycles, N, of stress range S, follows an identical format to the T-N relationship given in Equation (12), i.e. the equation for the representative S-N curve is

$$N S^m = K \tag{A.14}$$

Values for m and K are given in Table A.3 for a selection of chain link, connecting link and wire rope components in sea water.

Table A.3 — m and K values for representative S-N curves [35]

Component	m	K
Stud chain	3,0	$1,2 \times 10^{11}$
Studless chain (open link)	3,0	$6,0 \times 10^{10}$
Six/multi-strand wire rope (corrosion protected)	4,0	$3,4 \times 10^{14}$
Spiral strand wire rope (corrosion protected)	4,8	$1,7 \times 10^{17}$

It is permissible to use test data for a specific type of mooring line component in design. A linear regression analysis is then used to establish the S-N curve with the design curve located at least two standard deviations below the mean line. In the case of chain tests in air, the effect of sea water should be accounted for by a reduction of the fatigue life by 2,0 for studlink chain, and by a factor of 5,0 for studless chain.

It should be noted that the recommended reduction factor for stud chain is only applicable when the stud is perfectly fitted in the chain link. The fatigue life of a stud chain link is highly sensitive to variations depending on the tightening of the stud. When the stud gets loose, the scenario of stress distribution changes totally and this can lead to a significant reduction in fatigue life. These problems are avoided by using studless chain.

The fatigue safety factor γ_F for the S-N approach, in accordance with 10.5, is

$$\gamma_F = 5,0 \qquad \text{when } D_F \leq 0,8 \tag{A.15}$$

$$\gamma_F = 5,0 + 3,0\,(D_F - 0,8)/0,2 \qquad \text{when } D_F > 0,8 \tag{A.16}$$

where D_F is the adjacent fatigue damage ratio, which is the ratio between the representative fatigue damage D in two adjacent lines taken as the lesser damage divided by the greater damage, ($D_F \leq 1,0$).

The safety factors defined above are intended to allow the use of grouped lines while retaining a suitable level of safety. Further reference should be made to DNV DEEPMOOR [35].

For long-term mooring systems stress concentration factors due to bending of the chain links in the fairleads, bending shoes, guide tubes and chocks should all be considered in the fatigue analysis.

A.9.2 Fatigue resistance

A.9.2.1 Wire rope, chain and connecting links

No guidance is offered.

A.9.2.2 T-N curves

The recommended T-N curves are based on API RP 2SK [12] and Reference [13].

R3 chain is stronger than the same size ORQ chain by a factor of 1,057. The minimum breaking strength (MBS) of ORQ chain is given by

$$\text{MBS (in kilonewtons)} = 0{,}0211\, d^2\, (44 - 0{,}08d) \tag{A.17}$$

where d is the nominal chain diameter, in millimetres (mm).

Where shackles are required, offshore long-term mooring (LTM) shackles, including appropriate stress concentration factors, are preferred to Kenter shackles, pear links, C-links and D-shackles, which should be avoided.

Recent research indicates that mean tension has a significant influence on wire rope fatigue life and therefore should be included in the design curve equations. A mean tension of 0,3 MBS is considered to be representative for conventional mooring systems. For wire rope fatigue analysis, the following methods can be considered to account for the mean tension effect.

a) For each sea state, determine the mean tension and the corresponding design curve that is then used to calculate the fatigue damage for that sea state. This implies that different design curves are used for different sea states.

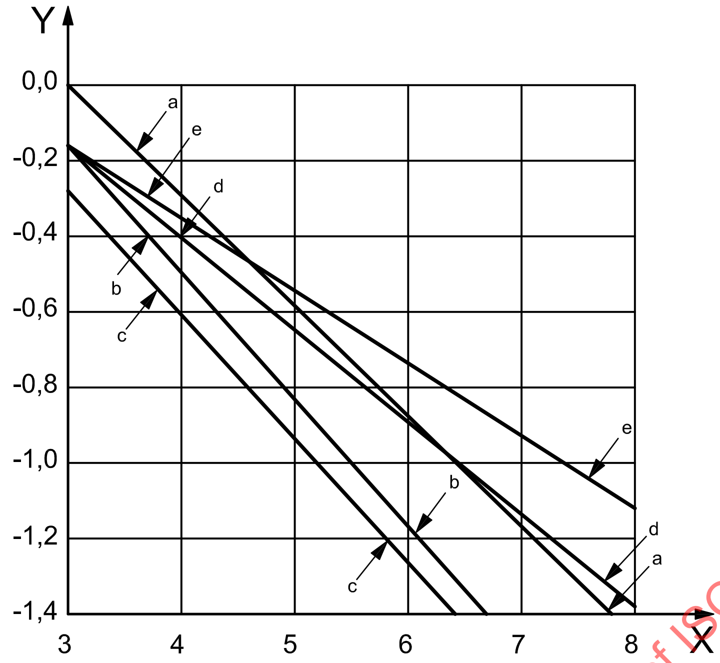
b) Determine the average mean tension for sea states causing significant fatigue damage and use the design curve for the average mean tension for all sea states.

c) Use the design curve for a mean tension of 0,3 MBS for conventional mooring systems.

Among the three methods, a) is the most accurate but requires more computational effort. If b) or c) are used, a sensitivity study should be performed to ensure that these simplified approaches produce conservative predictions.

The T-N curves for wire rope are based on test data from [25].

Figure A.13 presents the T-N fatigue design curves for chain, connecting links, six/multi-strand rope, and spiral strand rope. The two curves for wire ropes are for a mean tension equal to 30 % of the reference breaking strength, i.e. = 30 % of MBS.



Key

X log 10 (number of cycles)

Y log 10 (tension range/reference breaking strength)

[a] Chain, studlink.

[b] Chain, studless.

[c] Connector, Kenter link.

[d] Six/multi-strand, mean load 30 % MBS.

[e] Spiral strand, mean load 30 % MBS.

Figure A.13 — Fatigue design curves for chain, connecting links and wire rope

A.9.2.3 Tension-tension (T-T) fatigue

No guidance is offered.

A.9.2.4 Bending-tension (B-T) and free bending fatigue

Data for bending-tension fatigue of chain and wire rope are insufficient for generating design curves. In the absence of fatigue design data, precautionary measures should be taken to avoid mooring failure due to bending-tension fatigue. For example, the bend diameter to rope diameter ratio should be large enough to avoid excessive bending, see Table A.4.

Table A.4 — Guidance on bending-tension fatigue life compared with tension-tension fatigue life

Wire rope type	Bend diameter ÷ rope diameter (ratio)	B-T fatigue life/T-T fatigue life %
Six strand	20	3
	70	8
Multi-strand	20	5
	70	15
Spiral strand	20	0,5
	70	1,5

The portion of mooring line in direct contact with a fairlead should be regularly inspected. This portion should also be periodically shifted to avoid constant bending in one area. A study comparing the bending-tension and tension-tension fatigue lives of mooring lines on a semi-submersible under typical North Sea environment provided the data given in Table A.4. This information provides reference values for establishing an operation policy to avoid excessive bending-tension fatigue for wire ropes. Because of the complexities of B-T fatigue condition, the guidance given in Table A.4 should be treated with caution and the fatigue safety factor given in 10.5 should be increased accordingly.

For bending-tension of chain, the portion of mooring line in direct contact with a fairlead should also be regularly inspected and shifted to avoid constant bending in one area. In general, the worst load case is to have a horizontal link subjected to bending-tension over a shallow groove. This often results in very high stress in the stud weld region. Therefore, fairleads should be shaped and sized to avoid this type of unfavourable bending of chain links. Limited fatigue T-N tests of chains over a five-pocket fairlead indicate a fatigue life of 5 % to 20 % for bending-tension compared to the T-T fatigue life. A seven-pocket fairlead design generally gives much improved B-T fatigue life.

A.9.3 Fatigue analysis procedure

A.9.3.1 General

No guidance is offered.

A.9.3.2 Cumulative fatigue damage

No guidance is offered.

A.9.3.3 Fatigue damage assessment

A.9.3.3.1 General

In the case where n follows a Rayleigh distribution and the T-N curve is consistent with the form of Equation (12), the annual fatigue damage accumulated in an individual MDS may be computed as

$$D_i = \frac{n_i}{K} E\left(T^m\right) K_i \tag{A.18}$$

where

n_i is the number of cycles associated with MDS$_i$, years^{-1};

$E(..)$ is the expected value;

T, m and K are as defined in 9.2.2.

A.9.3.3.2 Tension range calculation

No guidance is offered

A.9.3.3.3 Combining wave-frequency and low-frequency tensions

The basis for the dual narrow-banded correction factor in 9.3.3.3.4 is discussed in [57].

Cycle-counting for fatigue analysis, including rainflow counting, is described in [66].

Figure A.14 presents fatigue damage, D, in a stationary sea state calculated using the cycle estimating algorithms,

a) simple summation,

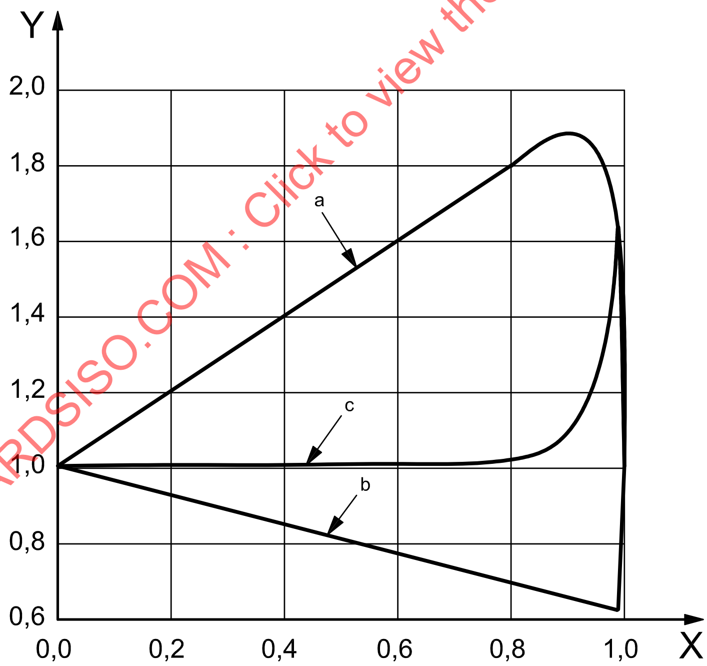
b) combined spectrum, and

c) dual narrow-banded spectrum,

compared to the damage, D_{rfc}, from the rainflow counting method.

The calculated fatigue damage ratios D/D_{rfc} are given in Figure A.14 as a function of λ_{Li}, as defined in Equation (21).

These results were obtained with a T-N fatigue curve with m = 3,0 and a factor of 10 between the mean zero-crossing periods of the tension variations in MDS, due to low-frequency and wave-frequency excitation, respectively.



Key

X λ_{Li}, see Equation (21)

Y fatigue damage ratio, D/D_{rfc}

[a] Combined spectrum.

[b] Simple summation.

[c] Dual narrow banded.

Figure A.14 — Fatigue damage ratio with various cycle counting algorithms

A.9.3.3.4 Accounting for mean values of tension in wire rope

See A.9.2.2.

A.10 Design criteria

A.10.1 Floating structure offset

A.10.1.1 Drilling operations

The mean offset should be controlled under the drilling operating condition because of its relevance to the mean ball or flex joint angle of the drilling riser. The allowable mean offset should be determined by a drilling riser analysis; allowable mean offsets depend on many factors such as water depth, environment, and riser system.

The maximum offset during drilling operations should be limited to prevent damage to the mechanical stop in the ball or flex joint below the drilling riser. The allowable maximum offset should be determined by a drilling riser analysis.

Further reference can be made to API RP 16Q[16].

A.10.1.2 Production operations

There are basically four types of production risers:

a) rigid riser,

b) flexible riser,

c) hybrid riser, and

d) riser integrated with single point mooring.

Rigid risers are tensioned from the structure and can be either integral or non-integral. An integral top tensioned riser is a multi-bore riser in which all fluid connections are made with a single coupling. A non-integral top tensioned riser consists of individual stands of pipe with individual connections for each flow path. A flexible riser consists of flexible pipe that hangs in a catenary from the FPS to the sea floor. Rigid and flexible risers can be combined into a hybrid production riser. A hybrid riser consists of a buoyant stand of rigid riser terminating at a point below the water surface. Flexible risers span the gap between the top of the rigid riser and the structure. The fourth type of production riser includes those that integrate the production risers with the single point moorings such as a CALM or SALM system.

The offset limits for the floating structure under the ULS and SLS design situations should be determined by a production riser analysis in conjunction with mooring analysis. Maximum allowable offsets for rigid risers normally fall in a range of 8 % to 12 % of water depth. This offset limitation often dictates that rigid production risers be disconnected during severe storms.

Maximum allowable offsets for deepwater flexible risers normally range from 10 % to 15 % of water depth, depending on the riser configuration. The maximum allowable offsets for shallow water flexible risers normally range from 15 % to 30 % of water depth. Flexible risers are usually designed to survive the maximum design environment while remaining connected to the structure.

A.10.1.3 Tender operations

Offsets for structures moored in proximity to another installation are limited by clearance requirements. The offsets under the intact, redundancy check and transient conditions should be limited to avoid contact of the structure or its mooring with the nearby installation.

A.10.2 Line tension limit

The criteria in Table 5 apply to both ULS and SLS. This is a departure from the previous practice where a lower tension limit was recommended for SLS. The rationale for the departure is as follows.

— For operations such as drilling and well intervention where the maximum operating environment is significantly lower than the ULS design situation, tensions need to be checked for the ULS design situation only. If the ULS criteria are met, tension is not a concern for the milder maximum operating environment.

— For operations such as certain floating production operations where production will continue under the ULS design situation, the ULS design situation and the SLS design situation are the same, and the same tension criteria should apply.

A.10.3 Grounded line length

It should be noted that insufficient line length can introduce vertical actions at the anchors.

A.10.4 Anchoring systems

A.10.4.1 General

The design of the anchoring system should ensure that, allowable limits of stress, displacement and fatigue in the anchor, and cyclic degradation in the surrounding soil, are not exceeded during and after installation. The anchoring system above the sea floor should include provisions for inspection and maintenance. The extent of inspection, timing of the inspection, and maintenance should be commensurate with the redundancy relative to overall safety and performance.

A number of design and installation issues for driven piles, suction piles, and plate anchors, all of which are capable of resisting vertical forces, are addressed in A.10.4.3 and A.10.4.4. These issues include anchor capacity evaluation, structural design, fabrication, handling and transportation, installation, and pull testing.

Some of the technological aspects of the design of suction piles and plate anchors are still under development. Specific and detailed recommendations are given in this annex to the extent currently possible. General statements are also used to indicate that considerations should be given to some particular aspects, and references are given for further guidance. Recommendations for design and installation of plate anchors in clay are given in Reference [34].

A.10.4.2 Drag anchors

A.10.4.2.1 General

Drag anchors are primarily used for mobile moorings and, in such cases, the design safety factors for anchors are substantially lower than those for line tensions. The rationale is to allow the anchor to move instead of the mooring line breaking in the event of mooring overload. Anchor movements of the most heavily loaded lines would normally cause favourable redistribution of the mooring line tensions. This is expected to help the mooring system survive environmental actions exceeding those from the ULS design situation.

Evaluation of anchor holding capacity is addressed here and in DNV-RP-E301[33].

Drag anchor technology has advanced considerably in recent years. Engineering and testing indicate that the new generation of fixed fluke drag anchors develops high holding power even in soft soil conditions. A high efficiency drag anchor is generally considered to be an attractive option for mooring applications because of its easy installation and proven performance. The anchor section of a mooring line can be preinstalled and test loaded prior to floating structure installation.

The holding capacity of a drag anchor in a particular soil condition represents the maximum horizontal steady pull that can be resisted by the anchor at continuous drag. This includes the resistance to the chain or wire rope in the soil for an embedded anchor, but excludes the friction of the chain or wire rope on the sea floor.

Drag anchor holding capacity is a function of several factors, including the following:

— Anchor type: fluke area, fluke angle, fluke shape, anchor weight, tripping palms, stabilizer bars, etc. Figure A.15 shows drag anchors commonly used by the offshore industry.

— Anchor behaviour during deployment: opening of the flukes, penetration of the flukes, depth of burial of the anchor, stability of the anchor during dragging, soil behaviour over the flukes, etc.

Furthermore, a long drag distance may be required for an anchor to reach full penetration and develop the ultimate holding capacity. This may be acceptable for anchoring a drill rig in an open water location but is likely to be unacceptable for a production location where the seabed is congested with subsea installations.

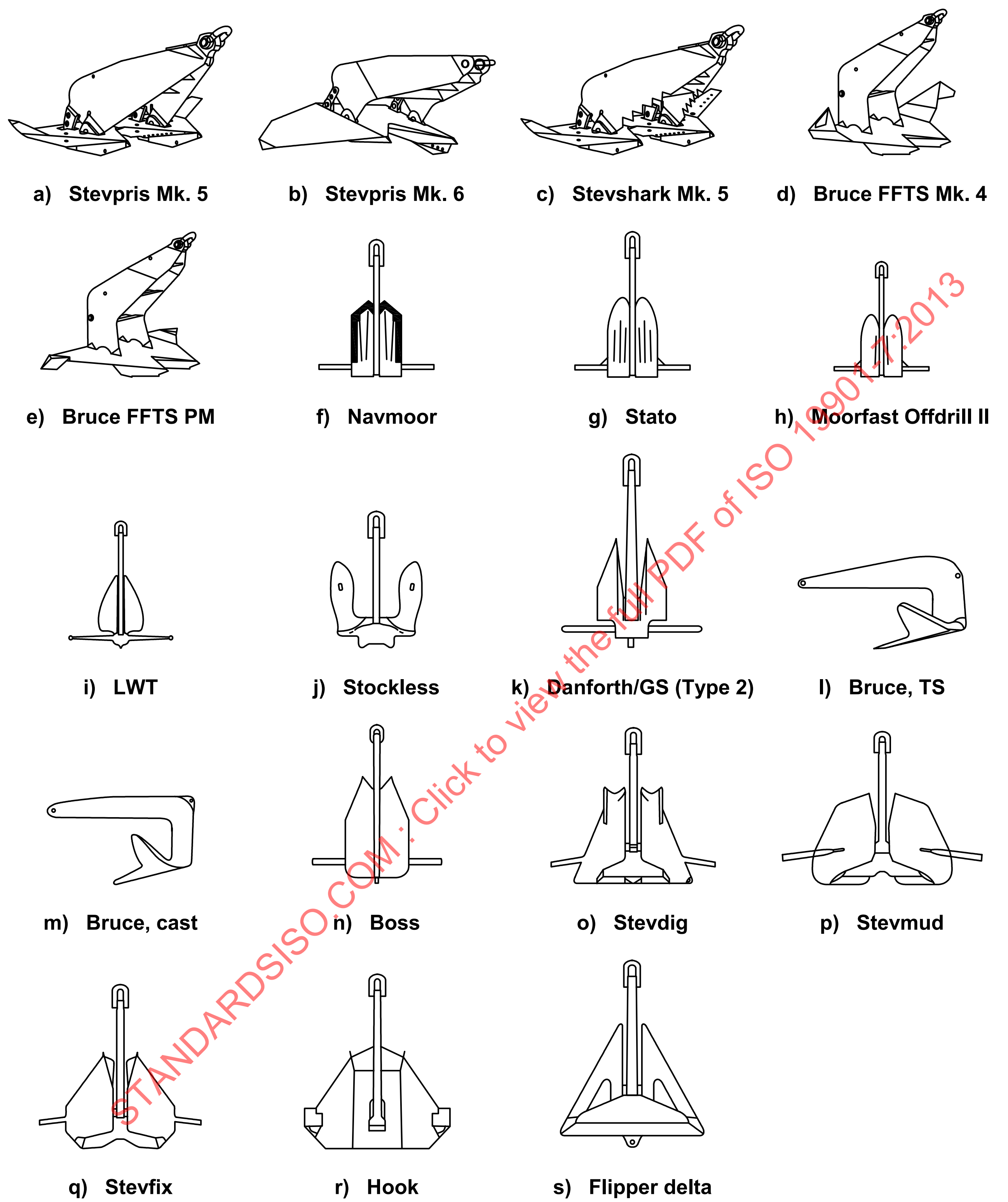
Due to the wide variation of these factors, the prediction of an anchor's holding capacity is difficult. Exact holding capacity can only be determined after the anchor is deployed and test loaded.

Anchor performance data for the specific anchor type and soil condition should be obtained if possible. In the absence of credible anchor performance data, Figures A.17 and A.18 may be used to estimate the holding power of anchors commonly used to moor floating vessels. Note that the holding capacity curves in Figures A.17 and A.18 do not include a design safety factor.

Figures A.17 and A.18 are reproduced from Reference [12], except that the holding capacity curves for the Moorfast (or Offdrill II) and Stevpris anchor were upgraded. The upgrading of these two curves was based on model and field test data and field experience acquired in recent years. The design curves presented in these two figures represent in general the lower bounds of the test data. The tests used to develop the curves were performed at a limited number of sites. As a result, the curves are for use in generic soil types such as soft clay (i.e., normally consolidated clay with undrained shear strength increasing monotonically with depth) and sand.

Recent studies indicate, however, that several parameters such as soil strength profile, lead line type (wire rope versus chain), cyclic actions, and anchor soaking can significantly influence anchor performance in soft clay. Also, some high efficiency anchors have demonstrated substantial resistance to vertical actions in soft clay. Furthermore, there are new versions of high efficiency anchors that are not covered by Figures A.17 and A.18, details of which are given in the following clauses.

As Figures A.17 and A.18 only provide anchor holding capacity estimates, more detailed analyses are needed if uncontrolled anchor drag cannot be tolerated in congested subsea locations where it may cause damage to existing subsea installations. If it is impractical to apply an installation tension required to completely avoid anchor drag, it may be necessary to demonstrate that the extent of anchor drag that can occur will not impinge on the existing subsea installations in the area.



NOTE The anchors named here are examples of suitable products available commercially. This information is given for the convenience of users of this part of ISO 19901 and does not constitute an endorsement by ISO of these products.

Figure A.15 — Drag anchors

A.10.4.2.2 Effect of soil shear strength gradient in clay

Centrifuge test data, as well as results from analytical studies using a calibrated drag embedment anchor prediction tool, indicate that a more or less linear relationship exists between the anchor holding capacity and the shear strength gradient of the clay [105]. However, significant deviations from this linear relationship are observed when the shear strength seabed intercept and/or the sensitivity of the clay are varied in addition to the shear strength gradient. In general, the effect of the various parameters on the anchor holding capacity in clay accentuates with increasing degree of mobilization of the anchor capacity. Of course, this relationship varies also with the anchor type and anchor size.

Due to the complexity of the problem, a reliable, calibrated prediction tool that can take all influencing parameters into account should be used to establish a basis for design of drag embedment anchors (see Reference [153]).

A.10.4.2.3 Effect of lead line type in clay

Field tests and analytical studies indicate that, in soft clay, when the lead line is wire rope, an anchor can penetrate deeper and give significantly higher holding capacity than when a chain lead line is used. For the limited cases studied, an anchor connected to wire rope provided 15 % to 40 % higher holding capacity than the same anchor connected to chain. This is in good agreement with the results from a full scale test programme. It should be noted that the studies were limited to high efficiency anchors in soft clay with a fairly constant shear strength gradient. A side effect is that the required anchor installation tension is reached with less drag if a wire lead line is used instead of a chain lead line.

A.10.4.2.4 Effect of cyclic loading in clay

Cyclic loading affects the static undrained shear strength (s_u) in two ways:

— during a storm, the rise time from mean to peak load can be about 3 s to 5 s (1/4 of a wave frequency tension cycle), as compared to 0.5 h to 2 h in a static consolidated undrained triaxial test, and this higher loading rate leads to an increase in the undrained shear strength and, consequently, in the anchor holding capacity;

— as a result of repeated cyclic loading during a storm, the undrained shear strength decreases; the degradation effect increases with increasing over-consolidation ratio of the clay.

The cyclic shear strength accounts for both these effects.

For more information about the prediction of cyclic loading effects, see DNV-RP-E301 [33], DNV-RP-E302 [34], and Andersen and Lauritzen [101]. A further development of these effects is presented in DNV-RP-E303 [104].

A.10.4.2.5 Effect of anchor soaking in clay

Soil set up due to thixotropy can lead to a significant increase in anchor holding capacity in a few hours or days after anchor installation, see for example results from temporary stoppage during instrumented field tests reported by Dahlberg and Strøm [146]. Over the subsequent weeks, soil set up due to thixotropy effects gradually increases in combination with soil consolidation (dissipation of excess pore water pressure).

Generally speaking, drag embedment anchors should therefore be installed without stoppage. A temporary stoppage before reaching the prescribed installation tension may prevent further anchor penetration if the increased tension required to restart the anchor after stoppage is higher than the pull available from the installation equipment. The consequence is that the long-term anchor capacity is no higher than that given by the installation tension of the initial step plus the increase due to post-installation effects (thixotropy/consolidation and cyclic loading effects). On the other hand, once the anchor starts to drag after a set up period this effect disappears completely.

In a design situation in which the anchor installation tension is intended to ensure stationkeeping of a floating structure without anchor drag, a safety factor should be applied to the predicted post-installation effects (set up and cyclic loading) and an adequate overall safety margin should be considered to determine the installation tension meeting such design requirement. In this case, the set up effect can represent a significant contribution to the total holding capacity, which should, however, be reduced for anchor penetration depths less than 2.5 fluke widths and be set to zero if the fluke penetration depth is very shallow (see further discussion in DNV-RP-E301 [33]).

A.10.4.2.6 Capacity in clay under inclined line loading

For deeply embedded drag embedment anchors (>2 to 2.5 fluke widths) the allowable uplift angle at the sea floor for ULS intact condition or redundancy checks can be set at values up to 20°, if proper anchor installation analyses have been carried out, showing that the uplift angle at the sea floor is significantly less than the uplift angle at the anchor padeye.

It is not advisable to apply a high uplift angle at the sea floor during the initial shallow penetration of the anchor; otherwise full penetration depth of the anchor is not achieved. After reaching a penetration depth >2 to 2.5 fluke widths, the line uplift angle at the seabed can be gradually increased. This issue is discussed in some detail in Reference [33].

Significant evidence supports the use of a non-zero uplift angle at the sea floor on drag embedment anchors that penetrate sufficiently deeply into soft clay. The following additional guidelines are proposed in this respect:

a) uplift angles at the sea floor should not be accepted for certain operations with mobile moorings where the soil conditions have not been thoroughly investigated or the anchor installation tension is insufficient to ensure deep anchor penetration.

b) the maximum uplift angle at the sea floor should be assessed according to the principles outlined above under the design situations for the ULS intact and redundancy checks.

c) a zero uplift angle should be maintained until the recommended minimum anchor penetration depth has been reached.

d) the anchor holding capacity should be reduced by a factor R, which is a function of the sea floor uplift angle, and accounts for the reduced friction due to shorter embedded line length. The R values in Table A.5 are applicable for Bruce FFTS Mark IV and Stevpris Mark V anchors:

Table A.5 — R values for Bruce FFTS Mark IV and Stevpris Mark V anchors

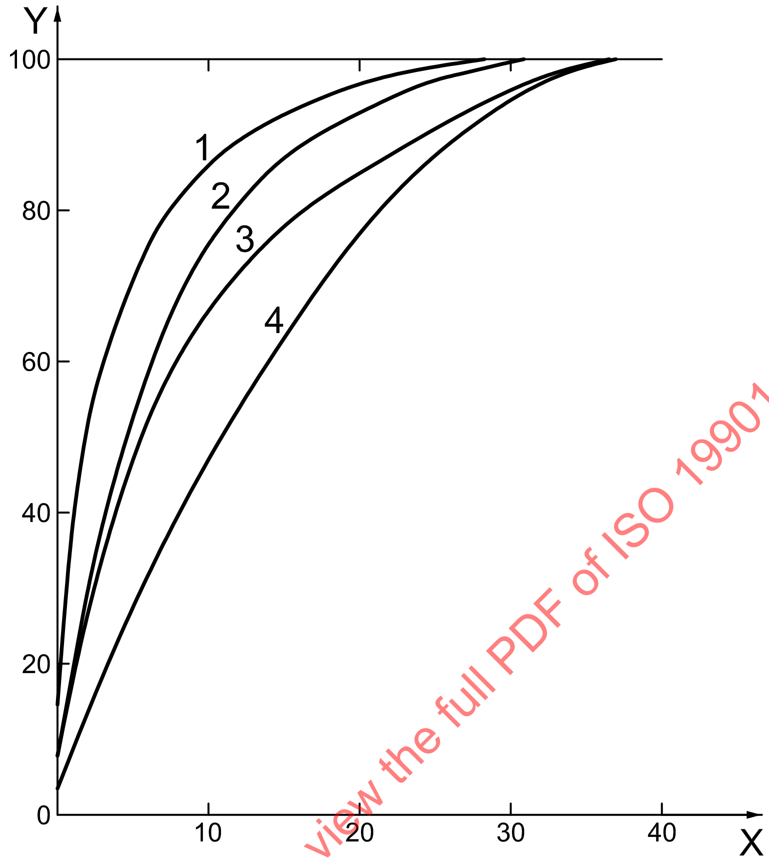
Sea floor angle °	0	5	10	15	20
R	1,0	0,98	0,95	0,89	0,81

When taut-leg mooring systems are utilized, the mooring lines lie at an initial angle to the sea floor and impose vertical and horizontal forces on the anchor at all times. As a consequence, drag anchors should not be used. A typical solution is to use anchor piles or plate anchors, for which design guidelines are provided in A.10.4.3 and A.10.4.4.

A.10.4.2.7 Drag distance and penetration depth in soft clay

Many factors affect drag-penetration depth, including site-specific soil data (soil stratigraphy, seabed shear strength, average shear strength gradient, soil sensitivity, etc.), and the size and type of anchor. For screening-level analysis, drag distance and penetration depth estimates from Reference [72] are presented in Figure A.16 and Table A.6, respectively. This information is valid for chain lead lines and shear strength gradients of 1,4 to 2,0 kPa/m. Deviation from this range can affect these values, especially the penetration depth estimates.

If the anchor design relies on further penetration to reach holding capacity, the additional drag to resist the design intact actions should not overload neighbouring lines.



Key

X drag distance/fluke length

Y percent of maximum capacity

1 stockless anchor (fixed)

2 hook anchor

3 anchor types Bruce, F FTS MK III/Bruce TS/Danforth/GS (type 2)[a]/LWT[a]/Moorfast/Navmoor/Offdrill II[a]/Stato/Stevmud/Stevpris MK III

4 anchor types Boss[a]/Flipper Delta[a]/Stevdig[a]/Stevfix/Stevin[a]

a Assumed based on geometric similarities.

Figure A.16 — Holding capacity vs. drag distance in soft clay [72]

Table A.6 — Estimated maximum fluke tip penetration [72]

Anchor type	Normalized fluke tip penetration (Fluke lengths)	
	Sands/stiff clays	Mud (e.g. soft silts and clays)
Stockless	1	3[a]
Moorfast Offdrill II	1	4
Boss Danforth Flipper delta GS (Type 2) LWT Stato Steyfix	1	4 ½
Sevpris MK III Bruce FFTS MK III Bruce TS Hook Stevmud	1	5

[a] Fixed fluke stockless.

A.10.4.2.8 New anchor design

New anchor designs and improvements to existing anchors continue to evolve. However, well controlled instrumented test and field performance data are insufficient for predicting the performance of many of these innovative high efficiency anchors, though results from such tests can be used to calibrate anchor prediction tools (see A.10.4.2.9). Just as important as the ultimate holding capacity is the ability to predict drag-penetration-tension relationships for mobilised loads which are much less than the ultimate holding capacity. In the absence of better information, the holding capacities of these new anchors can be conservatively estimated from the following equation:

$$H_n = H_s (A_n/A_s)^n \tag{A.19}$$

where

H_n is the holding capacity of new design;

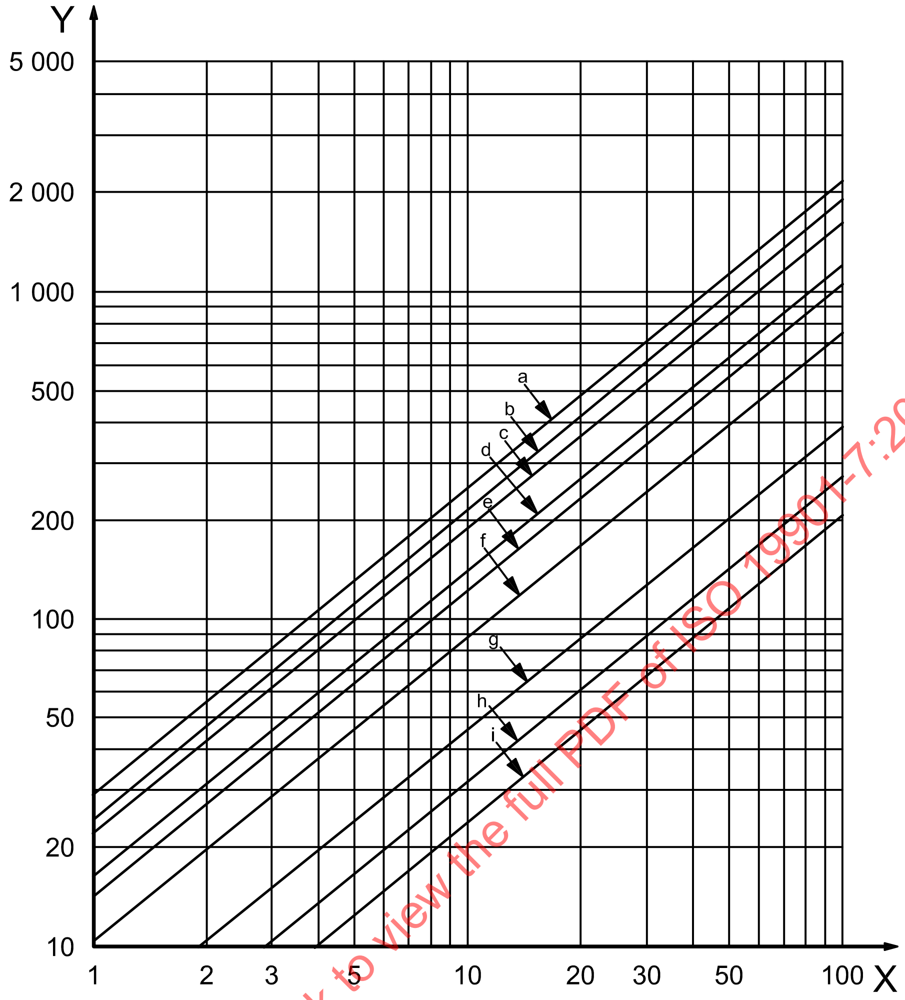
H_s is the holding capacity of reference design (e.g. Bruce FFTS Mark III or Stevpris Mark III in Figures A.17 and A.18) of the same weight;

A_n is the fluke area of new design;

A_s is the fluke area of reference design of same weight;

n is the 1,4 for commonly used high efficiency anchors.

The fluke area ratio A_n/A_s can be obtained from anchor manufacturers.



Key

X anchor weight (kips)

Y anchor holding capacity (kips)

Fluke angles set for mud sea floor condition as per manufacturer's specification.

NOTE 1 1 kip = 4,448 kN

NOTE 2 This figure was reproduced from Reference [72], except that the holding capacity curves for the Moorfast (or Offdrill II) and the Stevpris anchor were upgraded. The upgrading of these two curves was based on model and field test data and field experience acquired in recent years. The design curves in the figure represent in general the lower bounds of the test data. They reflect data valid for anchor designs as of 1987. New anchor designs have since been developed. However, performance data for these new designs were insufficient and therefore their design curves were not included. The design curves do not include a design safety factor.

NOTE 3 These anchors are examples of suitable products available commercially. This information is given for the convenience of users of this part of ISO 19901 and does not constitute an endorsement by ISO of these products.

[a] Bruce FFTS Mk. III, Stevpris Mk. III

[b] Navmoor, Stato, Boss.

[c] Bruce TS, Hook, Stevfix.

[d] Flipper Delta, Stevin, Stevdig

[e] Moorfast, Offdrill II.

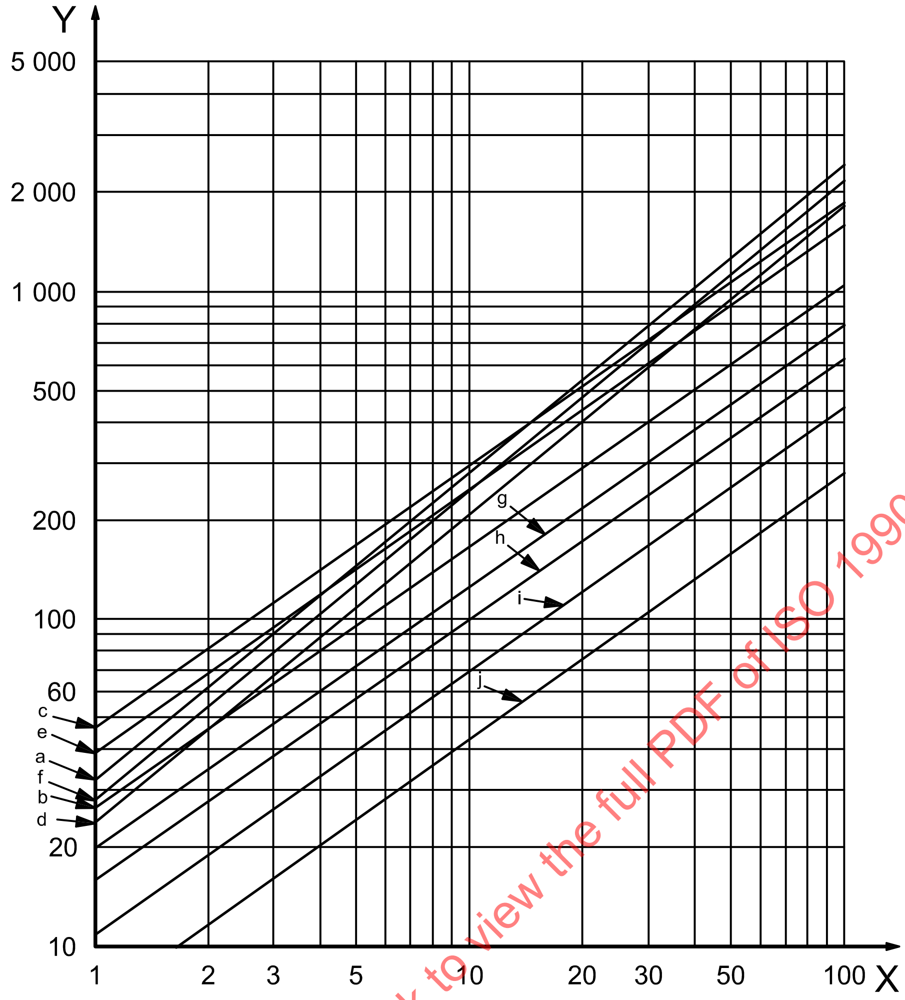
[f] Danforth, GS, LWT.

[g] Stockless, fixed fluke.

[h] Bruce, cast.

[i] Stockless, movable fluke.

Figure A.17 — Anchor-system holding capacity in soft clay



Key

X anchor weight (kips)

Y anchor holding capacity (kips)

Fluke angles set for sand sea floor condition as per manufacturer's specification.

NOTE 1 1 kip = 4,448 kN

NOTE 2 This figure was reproduced from Techdata sheet 83-08R, Reference [72]. The design curves in the figure represent in general the lower bounds of the test data. They reflect data valid for anchor designs as of 1987. New anchor designs have since been developed. However, performance data for these new designs were insufficient and therefore their design curves were not included. The design curves do not include a design safety factor.

NOTE 3 These anchors are examples of suitable products available commercially. This information is given for the convenience of users of this part of ISO 19901 and does not constitute an endorsement by ISO of these products.

[a] Navmoor, Boss.

[b] Stevin.

[c] Stevfix, Stevdig

[d] Stevpris, straight shank, Bruce TS

[e] Bruce, cast.

[f] Stato, 30° pulse angle.

[g] Danforth GS, LWT.

[h] Moorfast, Offdrill II, 20° fluke angle, Hook

[i] Stockless, 35° fluke angle.

[j] Stockless, 48° fluke angle.

Figure A.18 — Anchor system holding capacity in sand

A.10.4.2.9 Analytical tools for anchor performance evaluation

Analytical tools based on limit equilibrium principles for anchor embedment and capacity calculation in soft clay are available. These tools allow modelling of different anchor designs and provide detailed anchor performance information such as anchor movement trajectory, anchor rotation, mooring line profile below the sea floor, ultimate anchor capacity, etc. However, there are certain requirements for these tools to yield reliable predictions.

a) The analytical tool should be calibrated against results from high quality instrumented tests by field or by centrifuge testing performed on the type of anchor of interest.

b) The soil properties should be well known, which is not necessarily the case when designing and installing drag embedment anchors. Where the soil properties are uncertain, suitable upper and lower bound soil parameters should be established, and the anchor design should be based on the more conservative prediction.

c) Users should be aware of the tool's limitations and be familiar with mooring operations. For example, some tools typically show that the anchor penetration increases continuously, leading to higher and higher anchor holding capacity. In such cases, the user should consider limiting the drag distance for calculating the anchor holding capacity to a distance that does not result in unacceptable vessel excursions.

d) Empirical formulae or field experience, if available, should be used to support analytical predictions.

e) Analytical tools should be able to handle layered clay profiles. Some can handle layered clay profiles with sand layers of limited thickness while others cannot model layered soil profiles.

A.10.4.2.10 Anchor holding capacity in sand

No significant study on the behaviour of drag embedment anchors in sand has been carried out since the U.S. Navy's Study [72]. Anchors do not achieve deep penetration in sand and no uplift resistance can be relied upon from shallow penetration anchors in any soil conditions, i.e. the line uplift angle at the sea floor should be zero.

Contrary to anchors in soft clay, anchors in sand do not gain any additional capacity from post-installation effects due to thixotropy, consolidation or cyclic loading effects. This means that, in this case, the initial anchor installation tension should be set high enough to provide the required safety factor for the anchors and the mooring system. The installation tension should be high enough to provide a safety factor that accounts for the uncertainty in the load calculation. General principles for the design and installation of drag anchors in sand are provided the previous paragraphs and in Reference [33].

In dense sand, anchors that are installed by a MODU can in some cases still be visible at the sea floor after installation due to the limited capacity of MODU winches. In such cases, with shallow penetration anchors it is not recommended to assume that the anchors continue to penetrate upon overloading.

A.10.4.2.11 Anchor holding in soils other than soft clay and sand

Predicting anchor holding capacity in hard clay, calcareous sand, coral or rock sea floor and layered soil profiles is complex and is dependent upon the detailed soil/rock data for the location of each anchor cluster. In these soils/rocks the anchor penetration is often very shallow, which means that the same precautions as recommended for anchors in sand (A.10.4.2.10) should be followed.

A.10.4.2.12 Holding capacity generated by friction

The holding capacity generated by friction of chain and wire rope on the sea floor can be estimated using Equation (A.20).

$$P_{\mathrm{cw}} = f \times L_{\mathrm{cw}} \times W_{\mathrm{cw}} \tag{A.20}$$

where

P_{cw} is the chain or wire rope holding capacity;

f is the coefficient of friction between chain or wire rope and the sea floor;

L_{cw} is the length of chain or wire rope in contact with the sea floor;

W_{cw} is the submerged unit weight of chain or wire rope.

The coefficient of friction depends upon the nature of the sea floor and on the type of mooring line. Static (starting) friction coefficients are normally used to compute the holding power of the line and sliding coefficients are normally used to compute the friction forces on the line during mooring deployment.

If more specific data are not available, for chain and wire rope the generalized coefficients given in Table A.7 can be used for various sea floor conditions such as soft mud, sand, and clay. Guidance for calculation of the seabed friction is also provided in DNV-RP-E301 [33]. However, industry experience indicates that coefficients of friction can vary significantly for different soil conditions, and much higher values for the sliding coefficient of friction have been encountered.

Table A.7 — Mooring line friction coefficients

Mooring line	Coefficient of friction f	
	Static	**Sliding**
Chain	1,0	0,7
Wire Rope	0,6	0,25

A.10.4.3 Anchor piles

A.10.4.3.1 Driven anchor piles

A.10.4.3.1.1 Basic considerations

Driven anchor piles can be designed to provide adequate capacity for taut mooring systems. The design of driven anchor piles builds on a strong industry background in the evaluation of geotechnical properties and the axial and lateral capacity prediction for driven piles. The calculation of driven pile capacities, as developed for fixed offshore structures, is well documented in ISO 19902 [4]. The recommended criteria in ISO 19902 should be applied for the design of driven anchor piles, but with some modifications to reflect the differences between mooring anchor piles and fixed platform piles. Some of the guidance provided in A.10.4.3.2.3 for the structural capacity of suction piles can also apply to driven piles. The design of a driven anchor pile should consider four potential failure modes:

a) pull-out due to axial forces;

b) overstress of the pile and mooring line attachment padeye due to lateral bending;

c) lateral rotation and/or translation;

d) fatigue due to environmental and installation actions.

Factors of safety for holding capacity are provided in Table 7 in 10.4.3. Information on coupling between vertical and horizontal capacities can be found in A.10.4.3.2.2.4. Axial safety factors consider that the pile is primarily loaded in tension, and are therefore higher than for piles loaded in compression. As with other piled foundation systems, the calculated ultimate axial soil resistance should be reduced if soil set-up, which is a function of time after pile installation, is not complete before significant forces are imposed on the anchor pile.

As the lateral failure mode for piles is considered to be less catastrophic than the vertical mode, lower factors of safety are recommended in Table 7 for lateral pile capacity. Use of separate factors of safety for vertical and lateral pile capacities can be straightforward for simple beam-column analysis of, for example, mobile moorings (A.10.4.3.2.2.2.3.4), but more complex methodologies do not differentiate between vertical and lateral pile resistance. The safety factor should be in accordance with the guidelines of A.10.4.3.2.2.5.

A.10.4.3.1.2 Geotechnical and structural strength design

In most anchor pile designs, the mooring line is attached to a padeye located on the pile below the sea floor, to enhance the lateral capacity. As a result, the design should consider the mooring line angle at padeye connection resulting from the inverse catenary through the upper soil layers. Calculation of the soil resistance above the padeye location should also consider remoulding effects due to this trenching of the mooring line through the upper soil layers.

Driven anchor piles in soft clay typically have aspect ratios (penetration/diameter) of 25 to 30. Piles having such aspect ratios behave as if horizontally fixed in position at the pile tip, and consequently deflect laterally and fail in bending before translating laterally as a rigid body. Pile stresses should be limited by the provisions of ISO 19902 under ULS intact condition.

As argued in Reference [130], “static” p-y curves can be considered for the calculation of lateral soil resistance. “Cyclic” p-y curves can be more appropriate for fatigue calculations. A modification to the ISO 19902 p-y curves has been proposed in Reference [106] to ensure that lateral deflections are not over-predicted. Consideration should be given to degrading the p-y curves for deflections greater than 10 % of the pile diameter. In addition, when lateral deflections associated with cyclic loads at or near the mudline are relatively large (e.g., exceeding y_c as defined in ISO 19902 for soft clay), consideration should be given to reducing or neglecting the soil-pile adhesion (skin friction) through this zone.

The design of driven anchor piles should consider typical installation tolerances, which can affect the calculated soil resistance and the pile structure. Pile verticality affects the angle of the mooring line at the padeye, which changes the components of horizontal and vertical mooring line forces that the pile must resist. Underdrive affects the axial pile capacity and can result in higher bending stresses in the pile. Padeye orientation (azimuth) can affect the local stresses in the padeye and connecting shackle. Horizontal positioning can affect the mooring scope and/or angle at the vessel fairlead, and should be considered when balancing mooring line pretensions.

A.10.4.3.1.3 Fatigue design

A.10.4.3.1.3.1 Basic considerations

Anchor piles should be checked for fatigue caused by in-place mooring line forces. Fatigue damage due to pile driving stresses should also be calculated and combined with in-place fatigue damage. For typical mooring systems, fatigue damage due to pile driving is much higher than that caused by in-place mooring line forces.

A.10.4.3.1.3.2 In-place loading

A global pile response analysis accounting for the pile-soil interaction should be carried out for the mooring line reactions due to the fatigue sea states acting on the system. The local stresses that generate fatigue damage in the pile should be obtained by calculating a SCF (stress concentration factor), relative to the nominal stresses generated by the global analysis, at fatigue critical locations. These locations are typically at the padeye, at the girth welds between the padeye and the pile, and between subsequent pile cans.

The evaluation of SCFs for girth welds should account for local thickness misalignment at the weld. Equations for SCFs are given in References [107] and [108]. Note that the calculated SCF should be corrected by the ratio of the nominal thickness used in the pile response analysis to the lesser of the pile wall thicknesses joining at the weld. The SCF should be applied to the nominal pile stress range obtained at the weld location due to in-place actions, from which damage is calculated.

A.10.4.3.1.3.3 Installation loading

Dynamic actions due to hammer impact during pile installation induce fatigue damage on both padeye and pile girth welds. The evaluation of the cyclic actions involves the dynamic response of the pile-soil system due to the hammer impact. This requires a wave equation analysis per blow for a given hammer type and efficiency, pile penetration, and soil resistance. Various such analyses are conducted for representative pile penetrations. For each analysis, time histories of stress at the critical locations along the pile are developed, as well as the number of blows associated with the assumed penetration.

For either pile girth or padeye welds, fatigue damage calculations should be carried out at various weld locations using local stress range, derived from the wave equation analysis at the selected pile penetrations. The location of the girth weld should be determined by the pile makeup schedule. The local response should include the corresponding SCF effect. The number of cycles of the stress history per blow is obtained using a variable amplitude counting method, such as the reservoir [109] or rainflow methods.

A.10.4.3.1.3.4 Fatigue resistance

Applicable S-N curves depend on manufacturing processes and defect acceptance criteria. Typically, pile sections are welded by a two-sided SAW process and are left in the as-welded condition. For this case, the D-curve, as defined in Reference [110], can be used. Use of a higher S-N curve for this application, without additional treatment of the weld, should be demonstrated by relevant data. Use of weld treatment methods, such as grinding, may support the upgrading of the S-N curve, provided that

a) the grinding process is properly implemented,

b) weld inspection methods and defect acceptance criteria are implemented, and

c) pertinent fatigue data are generated to qualify the weld to a performance level higher than that implied by the D-curve.

A.10.4.3.1.3.5 Total fatigue damage and factor of safety

Once the fatigue loading and resistance are determined, fatigue damage due to in-place and installation actions can be evaluated using procedures similar to those described in Clause 9 and A.10.4.3.2.3.7. The total fatigue damage, D, should satisfy the following equation for the critical structural elements

$$D = F D_1 + F D_2 < 1 \tag{A.21}$$

where

F is the factor of safety, equal to 3.0;

D_1 is the calculated fatigue damage for Phase 1, i.e. installation (pile driving) phase and transportation phase, if significant;

D_2 is the calculated fatigue damage for Phase 2, i.e. in-service phase, during the service life (e.g. 20 years).

Further discussions on fatigue damage design for driven piles can be found in References [110] and [111].

A.10.4.3.1.4 Test loading of driven anchor piles

Driven pile installation records should demonstrate that the pile self weight penetration, pile orientation, driving records and final penetration are within the ranges established during pile design and pile driving analysis. Under these circumstances, test loading of the anchor to full intact storm load should not be required.

However, the mooring and anchor design should define a minimum acceptable level of test loading. This test loading should ensure that the mooring line's inverse catenary is sufficiently formed to prevent unacceptable mooring line slacking during storm conditions due to additional inverse catenary cut-in. Another function of the test loading is to detect severe damage to the mooring components during installation.

A.10.4.3.2 Suction anchors

A.10.4.3.2.1 General

A suction anchor can take many forms, ranging from a gravity base with skirts to a no-ballast suction anchor that resists all applied actions by soil friction, lateral resistance and reverse end bearing.

Generally, a suction anchor is technically feasible for soft to medium hard soils. For very soft soils such as those in some Mississippi Delta areas, a suction anchor extends so deep into the soil in order to reach competent load bearing material that it becomes unwieldy and difficult to handle. For very hard soils, it is sometimes not possible for the suction anchors to penetrate deeply enough to provide adequate in-place strength.

When lowered to the sea floor during initial installation, a suction anchor penetrates to a certain depth under its own weight and creates a seal to allow the suction operation to commence. Water is evacuated from inside the suction anchor with a submersible or surface vacuum pump through an umbilical attached to the top of the suction anchor. This causes the suction anchor to embed into the seabed to the design penetration. Additional ballast can then be placed in the suction anchor's upper chamber following completion of the embedment.

Some useful information for the design of suction anchors is provided in References [6], [41], [82], [89] and [104].

A.10.4.3.2.2 Geotechnical design

A.10.4.3.2.2.1 Basic considerations

The design of suction anchors for floating systems includes the following aspects: penetration and removal, capacity, and soil reactions or soil structure interaction analyses for structural design. In areas such as the Gulf of Mexico, where action effects of tropical cyclonic storms can exceed the capacity of the mobile mooring or mobile anchoring system, the design of suction piles should consider an anchor failure mode that reduces the chance of anchor pull-out. For site conditions where the presence of hard soil layers can limit suction anchor penetration, other anchor types should be considered instead.

The calculation of the geotechnical holding capacity of the anchor should be based on a best estimate of the soil properties. Anchor adequacy with respect to installation should be checked against upper bound soil strength properties. If faced with larger-than-usual scatter in the soil data, the designer should consider increasing the safety factors given in Table 7.

The impact of the mooring line geometry in the soil on anchor forces should be considered since the geometry can affect the relationship between the horizontal and vertical anchor forces. The inverse catenary of the mooring line in the soil can make the mooring line angle steeper at the anchor padeye than at the mudline. This steeper angle could result in a reduced horizontal force but an increased vertical force at the anchor padeye. Both an upper and lower bound inverse catenary should be checked to ensure the worst–case anchor loading is established.

A.10.4.3.2.2.2 Analysis methods

A.10.4.3.2.2.2.1 Penetration analysis

A.10.4.3.2.2.2.1.1 General

A typical penetration analysis includes the calculation of three quantities, for all penetration depths. These are:

— the penetration resistance exerted on the anchor by the soil;

— the required underpressure to allow anchor embedment;

— the critical pressure that can cause the soil plug to fail.

It is of paramount importance to properly estimate the underpressure required for the pile to achieve design penetration. Minimum underpressures are vital input parameters to the structural design of the anchor. Furthermore, the pumps used during installation should be capable of generating adequate underpressure.

A.10.4.3.2.2.2.1.2 Penetration resistance

The penetration resistance can be calculated as the sum of the side shear and end bearing on the side wall and any other protuberances. Protuberances include mooring and lifting padeyes, longitudinal or ring stiffeners, changes in wall thickness, mooring chain, launching skids, and others.

For an anchor in clay without protuberances and with a flat tip, the installation resistance, at a given tip penetration depth, z, can be calculated by Equation (A.22).

$$Q_{\text{tot}} = Q_{\text{side}} + Q_{\text{tip}} \tag{A.22}$$

$$Q_{\text{side}} = A_{\text{wall}} \left(\alpha_{\text{ins}}\, s_{u,\text{DSS}}\right)_{\text{AVE}}$$

$$Q_{\text{tip}} = \left(N_c\, s_{u,\text{tip}}^{\text{AVE}} + \gamma' \times z\right) \times A_{\text{tip}}$$

where

Q_{tot}	is the total penetration resistance;
Q_{side}	is the resistance along the sides of the pile;
Q_{tip}	is the resistance at the pile tip;
A_{wall}	is the sum of inside and outside wall areas embedded in soil;
A_{tip}	is the pile tip cross-sectional area (excluding contained soil);
α_{ins}	is the adhesion factor during installation (see Item a));
$s_{u,\text{DSS}}$	is the direct simple shear strength;
$\alpha_{\text{ins}}\, s_{u,\text{DSS}}$	is the side friction;
$(\alpha_{\text{ins}}\, s_{u,\text{DSS}})_{\text{AVE}}$	is the average side friction from mudline to depth z;
N_{c}	is the bearing capacity factor (see Item b));
$s_{u,\text{tip}}^{\text{AVE}}$	is the average of triaxial compression, triaxial extension, and DSS undrained shear strength at anchor tip level;
γ'	is the effective unit weight of soil;
z	is the tip penetration depth.

a) Adhesion factor during installation, α_{ins}

The adhesion factor during installation, α_{ins}, is usually defined as the ratio of remoulded shear strength over undisturbed shear strength, that is, as the inverse of the soil sensitivity. The adhesion factor can be determined by various methods but fall cone, UU triaxial, and miniature vanes (minivane) are the most common. The typical range of α_{ins} for Gulf of Mexico deepwater clays is 0.2 to 0.5.

There can be uncertainty in the sensitivity since it is influenced by the quality of the intact strength that it is related to. Alternatively, the side friction, $\alpha_{ins}\, s_{u,DSS}$, can be equated to the direct measurement of remoulded shear strength, through fall cone, UU triaxial, or minivane tests. The remoulded strength used in design should reflect both the directly measured value and the value derived from the intact strength divided by the sensitivity.

Some installation records have, however, shown that the interface shear strength mobilized during installation can, at a given depth, be less than $\alpha_{ins}\, s_{u,DSS}$. In cases where the full interface shear strength, $\alpha_{in}\, s_{u,DSS}$, cannot be mobilized along the anchor wall, such as when the anchor is painted or subjected to unusual surface treatment, a correction factor should be applied to α_{ins} to properly predict the penetration resistance [112], [113]. Ring shear tests, with the actual wall surface modelled in the tests, can be used to measure the actual interface shear strength.

b) Bearing capacity factor, N_c

The value of the bearing capacity factor N_c to be used to calculate the penetration resistance of the anchor tip or of a given protuberance depends on the shape of the protuberance and the ratio of the width of the protuberance over the embedment depth of the protuberance. Values of N_c ranging from 5.1 to 9.0 for round and strip footings are recommended in Reference [114].

Because the anchor wall thickness is usually small compared to the anchor diameter and the embedment depth, the pile tip is usually considered to be a deeply embedded strip footing with an associated N_c equal to 7.5.

The values of N_c to be used in Equation (A.22) are summarized in Table A.8.

Table A.8 — Recommended N_c factor

Purpose	Shape of area	N_c
Calculation of pile tip penetration resistance	Strip	7.5
Calculation of critical underpressure causing soil plug failure (see A.10.4.3.2.2.2.1.3)	Circular	6.2 to 9.0 depending on embedment ratio [114]
Calculation of penetration resistance of protuberances (see below)	Varies	5 to 13.5 [115]

A detailed example of the calculation of N_c is given in Reference [116]. Values of N_c different from those of Table A.8 are acceptable provided that they can be documented by appropriate modelling and test results.

c) Changes in penetration resistance due to protuberances

Equation (A.22) should be modified if protuberances are present. The change in penetration resistance due to the presence of mooring and lifting padeyes, longitudinal or ring stiffeners, mooring chain, launching skids, pile tip other than flat (i.e. bevelled) or any other internal or external protuberance should be considered carefully to assess the changes in friction and end bearing resistance caused by the protuberances. Most protuberances cause an increase in penetration resistance, except for internal ring stiffeners, which can cause a decrease in internal side friction if they are closely spaced [115], [117].

A.10.4.3.2.2.2.1.3 Required underpressure

The required underpressure, ΔU_{req} to embed the anchor can be calculated as follows:

$$\Delta U_{\text{req}} = \frac{Q_{\text{tot}} - W'}{A_{\text{in}}} \tag{A.23}$$

where

Q_{tot} is the total penetration resistance;

W' is the submerged weight during installation;

A_{in} is the plan view inside area where underpressure is applied.

A.10.4.3.2.2.2.1.4 Critical and allowable underpressures

The critical underpressure at a given depth, ΔU_{crit}, defined as the underpressure that causes a general reverse bearing failure at the anchor tip and large soil heave within the anchor, can be calculated at a given depth as follows:

$$\Delta U_{\text{crit}} = N_{\text{c}} \times s_{u,\text{tip}}^{\text{AVE}} + \frac{A_{\text{inside}} \times \left(\alpha_{\text{ins}} \times s_{u,\text{DSS}}\right)_{\text{AVE}}}{A_{\text{in}}} \tag{A.24}$$

where

A_{inside} is the inside lateral area of anchor wall.

In shallow water the critical underpressure should not exceed the water cavitation pressure.

The recommended allowable underpressure, ΔU_{allow}, defined as the maximum underpressure that should be applied to the anchor, can be calculated as the critical underpressure divided by an appropriate factor of safety. The minimum value of the safety factor is typically 1,5. Lower values can be acceptable provided that, during installation, the plug behaviour is monitored and it is confirmed that no plug failure occurred.

A.10.4.3.2.2.2.1.5 Soil plug heave inside anchor

The soil heave inside the anchor during installation can be estimated by assuming that a percentage of the clay volume displaced by the cross-sectional area of the anchor goes inside the anchor. This percentage depends on: anchor tip geometry, mode of penetration (i.e. self weight penetration vs. penetration by underpressure) [118]. It is commonly assumed the 50 % of the soil displaced by the cross-sectional area of the anchor tip goes inside the anchor during self weight penetration if the tip of the anchor is flat.

The final elevation of the internal plug surface depends on the wall thickness variations, internal soil plug stability, and spacing and type of internal stiffeners [118].

Soil heave should be accounted for in calculating the required pile stick-up and total length.

A.10.4.3.2.2.2.1.6 Items of special consideration

Sand layers, if present, should be given special attention. The penetration resistance in layered profiles consisting of interbedded sands and clays can be significantly higher than through clay, depending on the density, degree of cementation, grain size distribution, and thickness, spacing and depth of the sand layers.

The penetration rate through sand layers should be high enough to prevent excessive flow of water through the sand layers ahead of the anchor tip, as this may cause large plug heave.

A.10.4.3.2.2.2.2 Removal analysis

The geotechnical analysis should also consider anchor retrieval for the following cases:

a) Mobile moorings where anchor removal is needed for reuse of the anchor or to clear the sea floor. The suction pile retrieval procedures and analysis should account for the estimated maximum set up time;

b) Permanent moorings where local regulations require removal of the anchors after the structure has reached the end of its service life. The suction pile retrieval procedures and analysis should be based on full soil set-up.

c) Mobile or permanent moorings where installation tolerances are exceeded, a mooring line is damaged during installation, or for other contingencies.

The overpressure required to retrieve the anchor, $(\Delta U_{\text{req}})_{\text{retr}}$ can be calculated from Equation (A.25).

$$\left(\Delta U_{\text{req}}\right)_{\text{retr}} = \frac{Q_{\text{tot}}\left(t = t_{\text{r}}\right) + W'}{A_{\text{in}}} \qquad \text{(A.25)}$$

where

Q_{tot} $(t = t_{\text{r}})$ is the total soil resistance at time of retrieval, t_{r}. Time t = 0 is defined as the time at the end of penetration;

W' is the submerged weight during retrieval;

A_{in} is the plan view inside area where overpressure is applied.

When calculating the total soil resistance during retrieval, Q_{tot} $(t = t_{\text{r}})$, Equation (A.22) can be used with some modifications. It should be noted that the interface shear strength might be higher than its value during installation, due to soil set-up. A.10.4.3.2.2.3 gives guidance on assessing the increase in adhesion factor with time.

The designer should also be mindful of possible differences between end bearing resistance in tension and compression for protuberances. In addition, the maximum extraction pressure used should not be higher than the pressure causing soil plug failure.

The vessel removing the anchor is often capable of applying a lifting force on the anchor with the recovery line. This assistance can significantly reduce the required extraction pressure and should be included in the removal analysis. Therefore, if a load is taken by the lifting wire during retrieval, that load can be subtracted from the numerator in Equation (A.25).

The effect of the maximum extraction pressure on the steel structure of the suction pile should be considered (see A.10.4.3.2.3.4.5).

A.10.4.3.2.2.2.3 Holding capacity

A.10.4.3.2.2.2.3.1 General

Analysis and design tools to determine the capacity of suction anchors can be classified as one of three general methods [115]. These are, in order of detail:

— the finite element method (FEM) or other advanced numerical analysis;

— limit equilibrium or plastic limit analysis methods (models involving soil failure mechanisms);

— semi-empirical methods (highly simplified models of soil resistance including beam column models).

For the analysis and design of suction anchors for anchoring deepwater floaters, the central focus is the ultimate capacity of the suction anchor and not the load deflection behaviour.

It is recommended that suction pile design for permanent moorings use FEM, limit equilibrium techniques or limit analysis (see A.10.4.3.2.2.2.3.2 and A.10.4.3.2.2.2.3.3). For mobile moorings with mainly horizontal loads, semi-empirical methods such as beam-column analysis (see A.10.4.3.2.2.2.3.4) using load transfer-displacement curves (i.e. p-y, t-z, Q-z) described in ISO 19902 are also considered adequate if suitably modified. A method to modify p-y curves to account for the larger diameter of suction piles and to ensure lateral deflection is not overestimated can be found in Reference [106]. The merits and shortcomings of each method are discussed below.

A.10.4.3.2.2.2.3.2 The finite element method (FEM)

As discussed in Reference [115], the FEM is the most rigorous general method of analysis available for complex structural systems (including soil continua and soil-structure interaction). The FEM identifies the critical failure mechanism without prior user assumptions, provided an appropriate constitutive model is used. The FEM also has many advantages including the ability to include complex geometries, spatially varying soil properties, and non-linear constitutive behaviour with failure criterion. Major disadvantages include the required specialist knowledge of advanced numerical analysis and the large time investment to set up a model.

In ductile plastic systems (foundations in soft clays are usually in this category) the ultimate capacity of the system is independent of the sub-failure properties (e.g. Young's modulus, Poisson's ratio) [119]. It has been shown that carefully formulated and executed analyses give system load carrying capacities that compare favourably with the few exact, analytical solutions available [120].

FEM software programs are widely available and have been used to advantage for assessing specific suction pile configurations, matching the few experimental results available, and providing calibration of simpler models. As mentioned above, such analyses require special expertise and a significant investment in time and are therefore not yet well suited to parametric studies or conventional design iteration (such as are required for finding the optimum anchor line attachment point).

FEM analysis can, however, be warranted for complex load and/or soil conditions where little experience is available, or to gain insight on specific behavioural aspects of the foundation (i.e. assessment of pore pressure changes and effective stress path at any point within the soil mass).

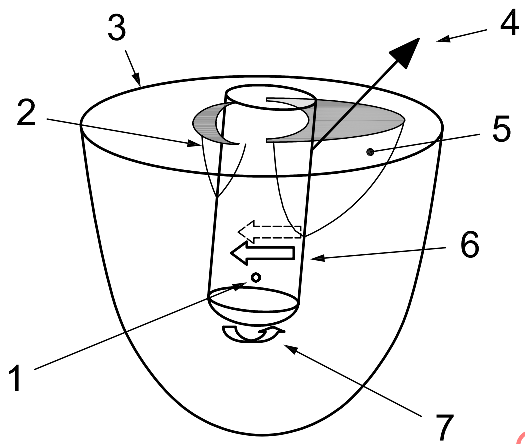
A.10.4.3.2.2.2.3.3 Limit equilibrium or plastic limit analysis methods

As discussed in Reference [115], these models are more approximate than FEM models but are generally much easier to use than general FEM programs. The methods involve estimating the ultimate capacity of plastic systems using assumed failure mechanisms. These mechanisms are typically based on a combination of experimental observation, more rigorous numerical or analytical studies, and engineering judgment. These methods can also include the ability to incorporate complex geometry and soil strength variability and do not require characterizing sub-failure behaviour.

Disadvantages of these methods are the approximate nature of the analysis and the difficulty of generalizing results, i.e. the need to calibrate the models to experiment or more rigorous analysis for specific structural configurations and soil profiles. For example, changes in soil strength profile, anchor geometry, load inclination, load attachment point, or load type (i.e. duration, frequency, ratio of cyclic to mean load component, etc.) can require a change in the basic geometry of the assumed failure mechanism.

In general there are two approaches that can be taken using assumed mechanisms; the limit equilibrium method and the plastic limit analysis method. In the limit equilibrium method, a failure mechanism is assumed, usually described in terms of one or more geometric parameters [121], [122]. The body force distribution, stress boundary conditions, and the stress or force distribution on failure surfaces are estimated, and a search is conducted to find the geometry that is closest to the equilibrium conditions. The plastic limit analysis method also uses an assumed failure mechanism with the added requirement that the mechanism satisfies kinematic constraints (i.e. incompressibility for a purely cohesive material, displacement continuity, etc.) [123], [124].

A possible failure mechanism is shown on Figure A.19. Other proposed mechanisms can be found in References [123], [125] and [126]. Depending on the failure mechanism, the anchor is shown to resist vertical uplift loads by self weight, skin friction, REB and/or shear and/or rotational failure at the pile tip, passive and active earth pressure, and soil flow around the pile.



Key

1 centre of rotation

2 active wedge failure zone

3 boundary of soil volume shown

4 applied load

5 passive wedge failure zone

6 flow around zone

7 tip rotational resistance

Figure A.19 — Three dimensional view of a possible failure mechanism

In some limit equilibrium methods, the circular area is transformed to a rectangle of the same area with the width equal to the diameter, and 3D effects are accounted for by side shear factors [121].

In general, both limit equilibrium and limit analysis methods give upper bound estimates of ultimate capacity such that minimizing the ultimate capacity with respect to the geometric parameters gives the “best” answer for the particular mechanism. However, the “best” answer is not always close to the ‘true’ answer depending on the assumed mechanism. In the limit equilibrium method the result is not a true upper bound if the mechanism does not satisfy kinematic constraints. A discussion of these methods is provided in Reference [119].

A number of existing computer programs implement these methods but there is no single general, industry accepted program or procedure.

Selected models have been shown to compare favourably with more rigorous FEM results for soft clay profiles and various anchor geometry and load attachment points [127].

Automated solutions using these approaches generally require much less input description and are much easier to use than general FEM programs. As a result they are well suited for conducting parametric studies and design iterations. However, as mentioned above, these solutions do not necessarily converge to correct capacity estimates even with great care and analyst skill, and results from different formulations can give significantly different answers. Thus, obtaining accurate results is greatly dependent on the analyst’s engineering judgment.

A.10.4.3.2.2.2.3.4 Semi-empirical methods: beam-column analysis

As discussed in Reference [115], these models are the most approximate, but generally are the easiest to use if computer programs with FEM, limit equilibrium or plastic limit analysis methods are not available. They are labelled semi-empirical to suggest that they incorporate the basic mechanics of a suction pile loaded to failure, but depend on a set of empirical rules to represent the soil resistance. These rules are typically less general than the methods discussed in A.10.4.3.2.2.2.3.2 and A.10.4.3.2.2.2.3.3. For example, they do not explicitly incorporate soil failure mechanisms, but instead represent the soil resistance as a load distribution varying along the boundary of the soil-pile interface. It is difficult to generalize such a load distribution for a wide range of soil profile types so a particular solution can apply, for instance, only to a normally consolidated strength profile. Rules for constructing these distributions are typically based on a combination of experimental and analytical results. In the so-called beam-column model, the soil is represented by uncoupled, non-linear soil springs along the pile boundary. The beam column method can provide estimates of the load displacement history up to and including the full capacity of the soil-pile system.

In the beam-column model, the soil resistance is represented by uncoupled, non-linear soil springs (p-y curves) which describe the sub-failure behaviour of the local soil resistance as well as the peak capacity [128] [129]. In the ISO 19902 p-y formulations for piles, the curves exhibit softening behaviour (reduced resistance with continued displacement) to account for the effects of cyclic loading [130]. It has been argued in Reference [131], however, that ultimate capacity estimates for piles, and thus presumably for suction piles as well, should be based on non-softening (static) p-y curves. In this model, the governing equations of a beam on an (non-linear) elastic foundation are solved iteratively until an equilibrium solution is found for a given value of the applied force. The user can gradually increase the force in subsequent steps until the solution no longer converges, a point which is interpreted as failure.

The beam-column model has been used by geotechnical engineers for almost 50 years for the analysis of laterally loaded piles. Hence it has the decided advantage of being a familiar tool. There are many beam-column programs in use, including general purpose programs where forces as well as non-linear springs can be prescribed at virtually any point on the pile, as well as special versions where non-linear spring construction is automated based on minimal soil property input. Thus, there could be an understandable tendency for engineers to select these programs for suction pile analysis. However, the user should be aware that these programs have significant limitations. As detailed in Reference [115], among the limitations, the conventional beam column models:

a) Ignore the fact that the resistance elements depend on the deformation mode and ignore the coupling between the resistance elements. This can lead to large errors, particularly for relatively short piles.

b) Do not include independent side shear resistance components on active and passive sides to model different relative shear displacements between the soil and the pile on the two sides.

c) Do not include the coupling between the horizontal and vertical soil resistance components along the pile sides and thus do not show the effect of inclined anchor forces. It is possible in principle to couple these elements (p-y and t-z curves), but this has only been done in special cases [132].

d) Require input that is not essential to the capacity assessment such as pile bending stiffness and sub-failure soil response and produce output that is of little interest for the analysis such as moment and shear profiles and load deformation response that are probably not very accurate. Because most piles are stiffened shells, the beam equations are of doubtful validity and are largely irrelevant with regard to stresses in the pile. A better pile model in these circumstances is actually a rigid body that can be approximated by setting the pile flexural rigidity (EI) to an arbitrarily large value (see A.10.4.3.2.3.5 for recommendations on structural design).

e) Require user intervention to determine the pile capacity. In most beam-column programs the ultimate capacity is determined by trial and error, gradually increasing or decreasing applied forces until the minimum force that produces numerical instability (interpreted to be the failure limit) is found.

f) Require special elements for rotational, vertical and horizontal tip resistance.

g) Do not explicitly include effects such as soil-pile interface roughness and loss of soil contact on the back side of the pile.

It is possible to formulate and implement a beam-column program that overcomes most of the above limitations. There seems to be little incentive to do so however, as other methods are available that are simpler to implement and can be especially tailored to suction pile analysis.

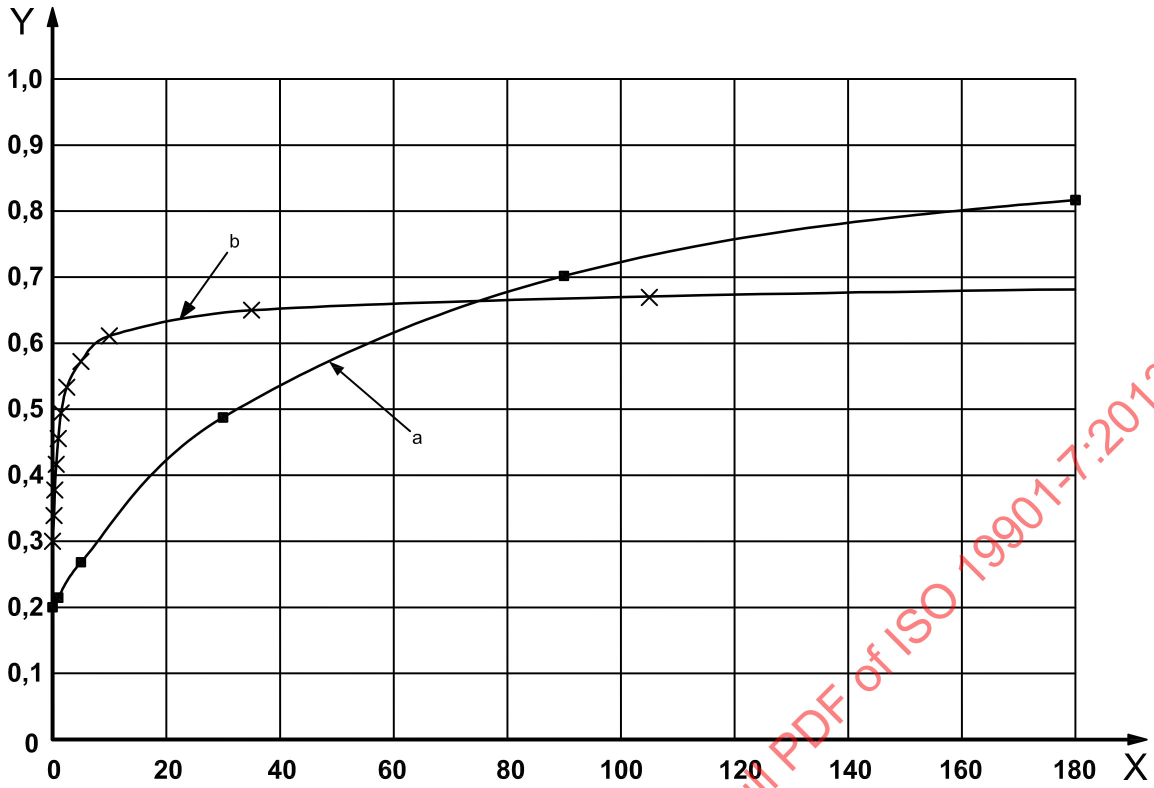
A.10.4.3.2.2.3 Increase of side friction with time

As described in A. 10.4.3.2.2.2.1.2, the side friction at a given depth can be calculated as $\alpha_{\text{ins}}\, s_{u,\text{DSS}}$. With the passage of time after installation, the side friction increases through soil thixotropic effects and pore pressure redistribution at the pile interface, i.e. set-up. Set-up effects are often addressed by estimating the change in the adhesion factor, α_{ins}, with time. Set-up mainly influences the vertical capacity, and to a lesser extent, the horizontal capacity of a suction pile [133].

The set-up process can be different for that part of the anchor penetrated by weight and that part penetrated by underpressure. For suction piles in highly plastic clays, the set-up time can be long and there can be a permanent loss of shear strength, whereby the ultimate side friction after full set-up is less than the original undisturbed shear strength (i.e. the adhesion factor, α_{ins}, is less than 1,0 after full set-up), both for that part of the anchor penetrated by weight and that part penetrated by underpressure.

Some researchers [134] have reported that the part of the anchor penetrated by underpressure is expected to typically have a shorter set-up time and a lower ultimate side friction after full set-up than the part penetrated by weight. Figure A.20 shows a typical soil set-up prediction graph for a large diameter suction anchor in typical Gulf of Mexico soils, and illustrates the current uncertainty in calculating soil set-up. The methods in References [133] and [135] are shown to illustrate the potential differences in set-up time and ultimate friction for different parts along the side of the anchor. The set-up along the part of the anchor penetrated by underpressure can occur much faster, but the permanent reduction can be larger. The method in Reference [135] was developed for driven piles with a ratio of diameter over wall thickness less than 40. The method in Reference [133] was proposed for penetration by underpressure. Both methods should be applied with caution outside the range of data used in their development. Other methods developed for driven piles include the one described in Reference [136]. There is no single industry-wide accepted set-up curve.

As with other piled foundation systems, the calculated anchor ultimate capacity should be reduced if soil set-up is not expected to be completed before significant forces are imposed on the anchor pile.



Key

X time after installation (days)

Y alpha = (side friction at time *t*) / (undisturbed shear strength)

a Bogard [135] — diameter = 6ft ; wall thickness = 1,8 in.; average curve.

b Andersen and Jostad [133] — alpha during installation = 0,3; alpha at 90% set-up = 0,65.

Figure A.20 — Example of increase in adhesion factor with time

Set-up can be addressed in various ways during design. The designer can ensure adequate anchor capacity if:

— the suction pile is designed with partial soil set-up;

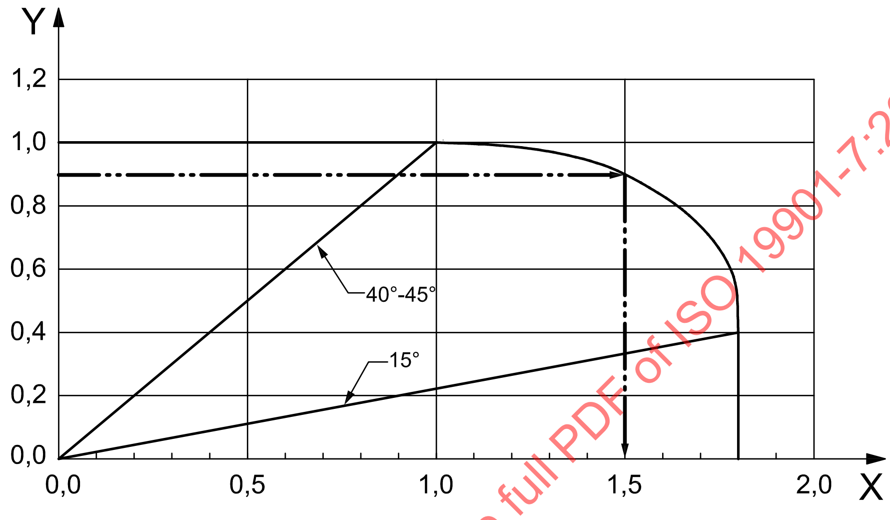
— the suction pile is installed well in advance of the floating structure hook-up to ensure adequate soil set-up when the mooring system experiences design actions;

— for a limited amount of time between installation of the mooring system and the start of production , reduced extreme action criteria may be assumed, based on suitable risk analysis.

A.10.4.3.2.2.4 Coupling between horizontal and vertical capacity

When a suction anchor resists design actions, the vertical and horizontal components of the anchor capacity are not mobilized independently. Coupling between vertical and horizontal capacities can be important in some cases. Studies have shown that for mooring line angles at the padeye between 15° and 45° (as measured from the horizontal), it can be non-conservative to neglect this coupling [121], [137].

The following discussion is for illustration only and therefore should not be used for design. The sample failure interaction diagram shown in Figure A.21 is typical of suction piles with a length to diameter ratio of 5, in soil with a linear increasing shear strength profile and low shear strength at the sea floor. The mooring padeye is located on the pile shell, about 2/3 of the way down from the pile top. In this example, the diagram shows that if the force is primarily vertical, with mooring padeye angles from 40° or 45° to 90°, the failure mode is controlled by vertical pull-out and 100 % of the vertical capacity is available. In a similar manner, if the force is primarily horizontal, with mooring padeye angles from zero to 15°, the failure mode is controlled by horizontal pull-out and 100 % of the horizontal pile capacity is available. In this case, the maximum horizontal capacity is equal to 1,8 times the vertical capacity. If, however, the mooring padeye angle is between 15° and 40°, less than maximum vertical and horizontal capacities are available. In the example shown, only 90 % of the vertical capacity is available; and the available horizontal capacity is reduced to 150 % of the vertical capacity, from an original 180 %.



Key

X H/V_{max}

Y V/V_{max}

V Vertical load component

H Horizontal load component

V_{max} Vertical ultimate capacity of the anchor, for purely vertical loads

NOTE 1 This interaction diagram is for illustration only and is not intended to be used for design.

NOTE 2 Mooring padeye angles are measured from the horizontal.

Figure A.21 — Example of failure interaction diagram

Examples of failure interaction diagrams can be found in References [138] and [139].

A.10.4.3.2.2.5 Factors of safety (FOS)

Factors of safety (FOS) for holding capacity, defined as the calculated capacity divided by the maximum anchor force from dynamic analysis, are provided in Table 7 (see 10.4.3) for axial and lateral forces. Information on coupling between vertical and horizontal capacities can be found in A.10.4.3.2.2.4. Axial FOS consider that the pile is primarily loaded in tension, and are therefore higher than for piles loaded in compression.

As the lateral failure mode for piles is considered to be less catastrophic than the vertical one, lower FOS have been recommended in Table 7 for lateral pile capacity. Use of separate factors of safety for vertical and lateral pile capacities can be straightforward for simple beam-column analysis of, for example, mobile moorings (see A.10.4.3.2.2.2.3.4), but more complex methodologies do not differentiate between vertical and lateral pile resistance.

The FOS used in design should be based on the failure mechanism controlling the capacity and not only on the mooring padeye angle. Although mooring padeye angle and failure mechanisms are related, other parameters such as soil profile, anchor geometry, and load attachment points are also important in determining failure mechanisms. For cases where axial pull-out controls, the minimum FOS should be as per Table 7, regardless of mooring padeye angle. For cases where lateral pull-out controls, the minimum FOS should be as per Table 7, regardless of mooring padeye angle. Equation (A.26) is proposed in order to provide a combined FOS for situations where neither the axial nor the lateral capacity controls the design. For a given geometry, padeye attachment point, and soil profile, the combined FOS can be calculated as follows:

If $\theta \leq \theta_{\text{lateral}}$, $FOS_{\text{combined}} = FOS_{\text{lateral}}$

If $\theta \geq \theta_{\text{axial}}$, $FOS_{\text{combined}} = FOS_{\text{axial}}$

If $\theta_{\text{lateral}} \leq \theta \leq \theta_{\text{axial}}$

$$FOS_{\text{combined}} = FOS_{\text{lateral}} + \frac{\theta - \theta_{\text{lateral}}}{\theta_{\text{axial}} - \theta_{\text{lateral}}} \times \left(FOS_{\text{axial}} - FOS_{\text{lateral}} \right) \tag{A.26}$$

where

FOS_{combined} is the combined FOS;

FOS_{lateral} is the lateral FOS from Table 7;

FOS_{axial} is the axial FOS from Table 7;

θ is the angle of mooring line from horizontal at pile attachment point;

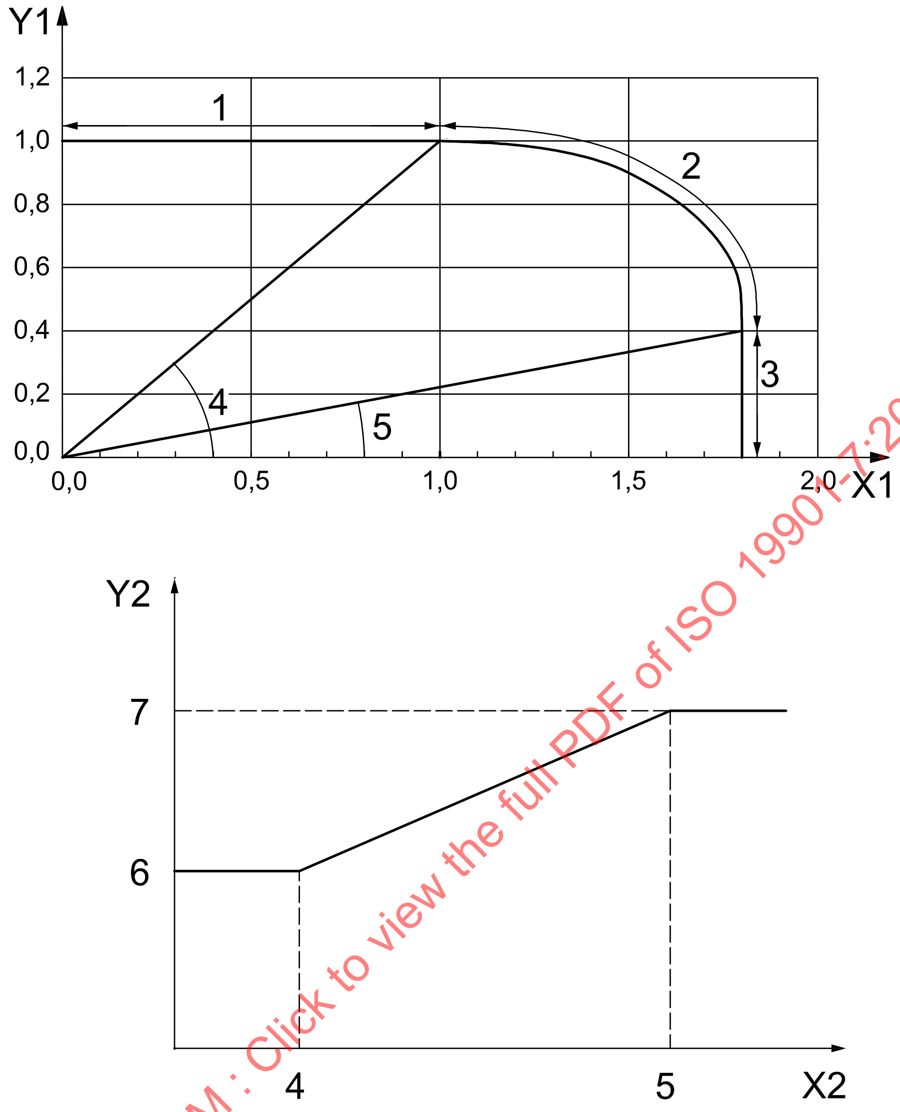
θ_{lateral} is the mooring line angle, measured from horizontal, below which the ultimate capacity is controlled by lateral capacity;

NOTE The lateral capacity is defined as the capacity under purely horizontal loads.

θ_{axial} is the mooring line angle, measured from horizontal, above which the ultimate capacity is controlled by axial capacity.

NOTE The axial capacity is defined as the capacity under purely vertical loads.

Figure A.22 illustrates the range of applicability of the various components of Equation (A.26).



Key

X1 H/V_{max}

X2 load angle (from horizontal)

Y1 V/V_{max}

Y2 factor of safety, $FOS_{combined}$

1 $FOS_{combined} = FOS_{axial}$

2 $FOS_{combined}$ as per Equation (E.5)

3 $FOS_{combined} = FOS_{lateral}$

4 θ_{axial}

5 $\theta_{lateral}$

6 $FOS_{lateral}$

7 FOS_{axial}

Figure A.22 — Calculation of required FOS as a function of failure mode

A.10.4.3.2.2.6 Other special considerations

a) Closed vs. open top

The top of the anchor should remain sealed throughout the life of the field, if the reverse end bearing (REB) at the anchor tip is to be relied upon in design. Note that, with increased soil set-up and side friction, the need for REB decreases and, thus, the requirement to maintain a sealed top cap. For anchors with essentially horizontal loading, a sealed top is not essential for ensuring capacity, and the top part can be removed after installation [140].

b) Strength anisotropy

Capacity calculations should be performed with anisotropic shear strength, including the effects of combined static and cyclic loading history.

c) Internal ring stiffeners

For large long-term loads and for suction piles that are not sealed at the top, the skin friction along the inside skirt wall is an important contribution to the capacity. The inside wall friction can be significantly lower than the original shear strength due to the disturbance during installation, especially if the anchor has internal stiffeners. In piles with ring stiffeners, clay from the upper part of the profile, and also water, can be trapped between the stiffeners and give low capacity at larger depth. In such cases, the compartment between the ring stiffeners can also act as a drainage channel [115].

d) Gapping

A gap can form on the outside at the active side (i.e. backside) of the anchor. There are uncertainties on how to predict gap formation, unless the clay is soft and with essentially zero strength intercept, in which case a gap is not expected to form. Therefore, one should make conservative assumptions with respect to whether there will be a gap or not. One should consider conservatively placing the load attachment point far enough below the optimal load attachment depth for the suction anchor top to move “backwards” (i.e. away from the direction of the mooring line) during loading to prevent gap formation.

e) Installation tolerances

The allowable installation tolerances (e.g. tilt and orientation) should be included in the capacity calculations, as tilt and out of plane loading can reduce the holding capacity of the pile.

f) Change in outer diameters

Variations in outer diameter with depth could reduce the outside interface strength; in general, designs with variations in outside diameters should be avoided.

g) Sand layers

Sand layers, if present, can have a significant effect on holding capacity. It should be ensured that the sand layers do not cause excessive drainage and pore pressure redistribution that could negatively affect the REB, particularly if the anchor is to resist long-duration loads.

h) Distance between installation locations

In the event that an anchor needs to be retrieved and re-installed, the determination of the minimum distance between the first location and the subsequent location should ensure that the soil disturbed during the first installation is not mobilized when the anchor resists the design load at the subsequent location.

i) Sustained action

The duration of sustained actions (e.g. creep under loop current action) and the period of cyclic loading should be considered, and the anchor capacities should be adjusted to account for these effects. Examples of capacity reduction as a function of action hold time for vertically loaded anchors in Gulf of Mexico clays can be found in Reference [141].

A combination of these considerations can be used to arrive at a suitable suction pile design. Due to the complexity of analysing the capacities of large permanent suction piles, a geotechnical expert should be consulted.

A.10.4.3.2.3 Structural design

A.10.4.3.2.3.1 Basic considerations

The purpose of this subclause is to provide guidance and criteria for the structural design of suction piles. Some of the guidance and criteria are also applicable to driven piles. Structural design for plate anchors is not addressed because it is typically performed by anchor manufacturers.

A.10.4.3.2.3.2 Fabrication considerations

The structural design criteria given in the following sections assume the suction pile has been fabricated to certain dimensions and tolerances. As a minimum, the following dimensions and tolerances should be specified in the suction pile fabrication specification in addition to the pile diameter and wall thickness schedule:

a) Pile length

Total pile length should be specified with a suitable tolerance. The minimum length so specified should be acceptable with respect to geotechnical design.

b) Out-of-roundness (OOR)

Out-of-roundness is the difference between the major and minor outside (or inside) diameters at any point along the length of the pile and should not exceed 1% of the nominal outside (or inside) diameter[4], [150]. The 1 % roundness value is the maximum OOR assumed in the buckling formulations given in ISO 19902 [4], API 2U [148] and DNV Classification Notes 30.1 [151].

For each cross-section, a minimum of two sets of two pairs of diameters each should be selected (i.e. eight points around the circumference of the pile). In Figure A. 23, these eight points would be A-E and G-C for the first set and F-B and D-H for the second set. Note that Figure A.25 also outlines a procedure to measure OOR on cans with their longitudinal axes horizontal. This technique, to a large extent, removes the effect on the OOR calculations of ovalization of the can due to gravity. Alternatively, OOR measurements can be made with the axis of the can vertical.

c) Circularity

Circularity is a measure of the pile wall's local deviation from its theoretical shape, in this case, an arc of the same radius as the pile. It is measured using a sweep gauge that has one edge cut to the theoretical inside or outside radius, as appropriate. The recommended sweep gauge arc length is 1/10th of the circumference of the pile. Measuring circularity ensures that dents, flat spots or other geometrical imperfections do not adversely affect the buckling resistance of the cylindrical pile wall during suction embedment.

As the sweep is moved around the circumference of the pile in a plane perpendicular to the axis of the pile, the gap between the sweep gauge and pile wall is measured. The acceptance tolerance can be determined from non-linear buckling analysis of the pile wall. The number of circumferences checked along the length of the pile should be sufficient to capture all potential dents or flat spots in the pile. A straightedge, held parallel to the longitudinal axis of the pile, should be used to survey the extent of any dents found using the sweep gauge; see References [4] and [151] for guidance for such measurements.

It is recommended that the dimensional control programme include OOR and circularity checks as part of the can forming process. Individual cans should not proceed to pile assembly until passing OOR and circularity requirements. The completed pile should also be subject to final OOR and circularity checks in addition to surveys of straightness and length.

d) Straightness

Generally, overall straightness should be consistent with the requirements of ISO 19902 [4]. Because of the relatively large diameter to length ratio of suction piles, such piles are not usually subject to a column buckling mode of failure that overall lack-of-straightness would exacerbate. However, local buckling failure is a possibility, hence the recommendation for the straightedge measurements in Item c).

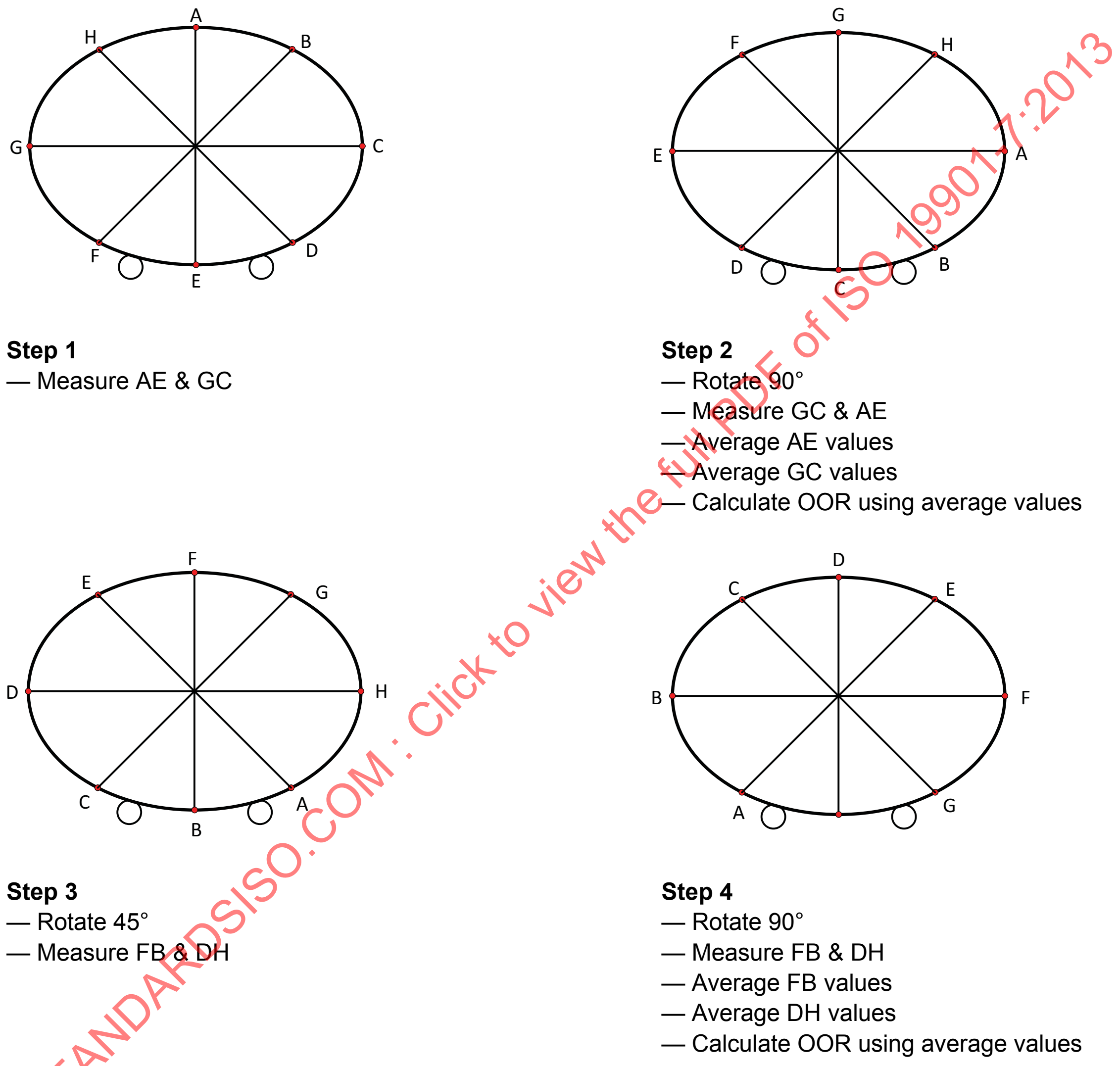


Figure A.23 — Procedure to measure pile out-of-roundness

A.10.4.3.2.3.3 Handling and transportation considerations

In order to achieve economic and weight control goals, it is not uncommon for suction piles to have large sections of thin-walled steel. These thin pile walls are especially vulnerable to damage by inadequate temporary support during handling operations in the fabrication yard. It is recommended that temporary supports be pre-engineered prior to handling and that rigging personnel be briefed on proper suction pile handling techniques.

Pile damage can also occur during loadout, transportation and offloading operations. Care should be taken when loading and unloading suction piles from their cradles to minimize side or vertical impact. Cradle design and fitment of the pile into its cradle should not invalidate the transportation design assumptions. For example, if nearly full cradle contact is necessary to keep the pile stresses generated during the transportation design event below allowable values, the actual pile fitment should match the assumptions or the cradle design should be modified accordingly.

A.10.4.3.2.3.4 Design situations

A.10.4.3.2.3.4.1 General

The suction pile structure should be designed to withstand the maximum actions applied by the mooring line, the maximum negative pressure required for anchor embedment, the maximum internal pressure required for anchor extraction, and the maximum actions imposed on the anchor during lifting, handling, launching, lowering and recovery. Fatigue lives of critical components and highly stressed areas of the anchor should be determined and checked against the required minimum fatigue design life.

A.10.4.3.2.3.4.2 Mooring actions on global anchor structure

The design situation that provides the maximum horizontal and vertical actions at the mooring padeye should be used for global structural design of the anchor. The soil reactions generated by the geotechnical analysis should be used in these calculations. Sensitivity checks should be performed to ensure that a design situation that induces an action of less than the maximum magnitude but applied at a more onerous angle at the padeye, does not control the design.

A.10.4.3.2.3.4.3 Mooring actions on anchor attachment

The mooring line attachment padeye or lug is a critical structural component. In order to meet fatigue resistance criteria, the padeye is often an integral cast lug and base structure. This avoids the use of heavy weldments which can result in a lower fatigue life. The padeye should be designed to satisfy both strength and fatigue requirements. The padeye should be designed for the controlling design actions with appropriate factors of safety. Designing the padeye for a maximum action equal to a factor times the break strength of the mooring line can lead to a significantly over-designed padeye, which can be difficult to integrate well with the anchor shell and back-up structures.

The mooring line padeye should be designed for the controlling design situation, and sensitivity checks should be performed to ensure that a design situation that induces an action of less than the maximum magnitude, but applied at a more onerous angle at the padeye, does not control the design. The orientation of the applied load at the padeye is affected by the inverse catenary of the mooring line, vertical misalignment due to anchor tilt, and rotational misalignment due to deviation from the target orientation. These factors should be properly accounted for.

A.10.4.3.2.3.4.4 Embedment actions

For anchor embedment, the estimated upper bound suction pressure required to embed the anchor to its design penetration should be used for the design of anchor wall and anchor cap structure. However, the maximum suction pressure used need not be higher than the suction at which internal plug uplift occurs.

A.10.4.3.2.3.4.5 Extraction actions

With respect to anchor extraction, there are two conditions that require evaluation:

a) Temporary condition – Extraction of a suction pile can be required for permanent moorings. For example, after all suction piles have been preinstalled along with the mooring lines, one of the mooring lines is accidentally dropped to the sea floor and damaged during the hook-up with the vessel. At this time, a decision to extract the suction pile and recover the mooring leg can be made. Typically, such situations can occur 30 to 60 days after the first suction pile has been installed.

   For mobile moorings, the suction piles are often extracted at the end of the current drilling or testing operation and reused at other locations.

b) Terminal condition – Suction piles for a permanent mooring can be extracted at the end of their service life.

The estimated maximum internal pressure required to extract the anchor for these two situations should be used for the design of anchor wall and anchor cap structure.

A.10.4.3.2.3.4.6 Transportation and handling actions

The suction pile structure and its installation appurtenances should be designed for the maximum actions induced by suction pile handling, transportation, lifting, upending, lowering, and recovery. The suction pile designer should interface closely with the installation contractor when determining appropriate design situations. Design of appurtenances for these design situations should meet the minimum requirements of ISO 19902.

A.10.4.3.2.3.5 Structural analysis

A.10.4.3.2.3.5.1 General

Pile analysis in accordance with ISO 19902 is appropriate for piles with diameter to thickness ratios (D/t) of less than approximately 100 to 120. For cylindrical piles with D/t ratios exceeding 100 to 120, it is recommended that a detailed structural finite element model be developed for the global structural anchor analysis to ensure that the anchor wall structure and appurtenances have adequate strength in highly loaded areas.

A.10.4.3.2.3.5.2 Space frame model

A space frame model generally consists of beam elements plus other elements needed to model specific structural characteristics. This is appropriate for piles with D/t ratios less than approximately 100 to 120 and for preliminary design of the top cap or padeye backup structures on large diameter piles (i.e. $D/t > 120$).

A.10.4.3.2.3.5.3 Finite element model

Finite element analysis is recommended for the global shell structure, top cap plate and supporting members and the padeye backup structure for piles with D/t greater than approximately 100 to 120. Complex shapes such as the padeye casting or welding should also be analysed by FEM.

A.10.4.3.2.3.5.4 Manual calculations

Manual calculations using empirical formulae and basic engineering principles can be performed where detailed finite element analysis is not needed.

A.10.4.3.2.3.5.5 Stress concentration factors

Stress concentration factors can be determined by detailed finite element analysis, physical models, and other rational methods or published formulae.

A.10.4.3.2.3.5.6 Stability analysis

Formulae for the calculation of the buckling strength of structural elements are presented in References [4], [148], [149] and [151]. As an alternative, buckling and post-buckling analysis or model tests of specific shell or plate structures can be performed to determine buckling and ultimate strength.

A.10.4.3.2.3.5.7 Dynamic response

Significant dynamic response is not expected for the anchor in its in-place condition, therefore anchor structures are often analysed statically. Transportation analysis, however, typically includes dynamic actions generated by the transportation vessel motions.

A.10.4.3.2.3.6 Structural design criteria

A.10.4.3.2.3.6.1 Design codes

In general, cylindrical shell elements should be designed in accordance with ISO 19902 [4] for D/t less than 120, or API 2U [148] or DNV RP-C202 [179] (DNV Classification Notes 30.1 [151] may also be used) when D/t exceeds 120, flat plate elements in accordance with API Bulletin 2V [149] or DNV Classification Notes 30.1, and all other structural elements in accordance with ISO 19902, as applicable. In cases where the structure's configurations or loading conditions are not specifically addressed by these codes, other accepted codes of practice can be used. In this case, the designer should ensure that the safety levels and design philosophy implied in this part of ISO 19901 are adequately met.

A.10.4.3.2.3.6.2 Safety categories

For a working stress design (WSD) approach, two safety categories can be identified: safety category A applies to normal design situations, and Safety Category B applies to rarely occurring design situations, as shown in Table A.8.

Table A.8 — Suction pile safety criteria

Design situation	Safety criteria
Maximum intact	A
Maximum one-line damaged	B
Anchor embedment	A
Anchor extraction (temporary)	A
Anchor extraction (terminal)	B
Handling / lifting / lowering / recovery	A
Transportation	B

A.10.4.3.2.3.6.3 Allowable stresses

For structural elements designed in accordance with WSD design standards (e.g. API RP 2A-WSD [152]), the allowable stresses recommended in those documents should be used for normal design conditions associated with Safety Criteria A. For extreme design conditions associated with Safety Criteria B, the allowable stresses may be increased by one-third

For shell structures designed in accordance with API 2U, a factor of safety equal to 1,67 Ψ is recommended for buckling modes for Safety Criteria A. For Safety Criteria B, the corresponding factor of safety is equal to 1,25 Ψ. The parameter Ψ varies with buckling stress and is defined in API 2U. It is equal to 1,2 for elastic buckling stresses at the proportional limit, and reduces linearly for inelastic buckling to 1,0 when the buckling stress is equal to the yield stress.

For flat plate structures designed in accordance with API 2V, the allowable stress is obtained by dividing the ultimate limit state stress by an appropriate factor of safety, which is 2,0 for Safety Criteria A and 1,5 for Safety Criteria B.

For cylindrical elements with D/t ratios exceeding approximately 100 to 120, it is recommended that global strength be analysed using FEM. Local buckling formulations for axial compression, bending and hydrostatic pressure given in ISO 19902 (for $D/t < 120$) and API 2U or DNV RP-C201 (DNV Classification Notes 30.1 may also be used) (for $D/t \geq 120$) are considered valid if due consideration is made for variable wall thicknesses and buckling length (which can extend below the mudline when performing suction embedment analysis).

The nominal Hencky-von Mises (equivalent) stress at a component's extreme fibre should not exceed the maximum permissible stress as given by Equation (A.27):

$$\sigma_A \leq \eta_i \, \sigma_y \tag{A.27}$$

where

σ_A is the nominal Hencky-von Mises stress;

η_i is the design factor for the specified design situation;

σ_y is the specified minimum yield stress of anchor material.

Values for the design factor for specified design situations determined using FEM are given in Table A.9.

Table A.9 — Design factors for finite element analysis

Design situation	Design factor η_i
Maximum Intact	0.67
Maximum Damaged	0.90
Anchor embedment	0.67
Anchor extraction (temporary)	0.67
Anchor extraction (terminal)	0.90
Handling / lifting / lowering / recovery	0.67
Transportation	0.90

The design factors recommended in Table A.9 are limits on the structure's primary stresses generated by the applied actions. Note that primary stresses are not self-limiting; i.e. primary stresses that exceed the yield strength of the material in question can result in failure.

Secondary stresses, on the other hand, can be developed by local structural discontinuities or by constraint of adjacent parts; such stresses are self-limiting. In some cases, it can be acceptable to exceed material yield for secondary stresses in elastic design. When local yielding is exceeded, the material should have sufficient ductility to enable adequate redistribution to adjacent areas of the structure.

A.10.4.3.2.3.7 Fatigue design

A.10.4.3.2.3.7.1 Fatigue analysis

In-place fatigue of the anchor structure is caused by tension-tension cyclic loading of the mooring line attached to the anchor. The fatigue analysis for suction anchors is similar to that for the mooring system, as discussed in Clause 9. The major differences are as follows.

a) The S-N (stress range vs. number of cycles to failure) approach is recommended for suction anchor fatigue analysis instead of the T-N (normalized tension range vs. number of cycles to failure) approach used for the mooring system. Appropriate S-N curve formulations that include the effect of member thickness should be utilized.

b) Normalized tension ranges from mooring fatigue analysis are converted to stress ranges for suction anchors using highly refined FEM.

Fatigue should be checked not only at joints, but also at any details with high SCF, e.g. the padeye casting at the base of the lug and in the eye.

A.10.4.3.2.3.7.2 Fatigue life requirement

The fatigue safety factor for anchor piles should be in accordance with 10.5.

A.10.4.3.2.4 Installation

In order to verify that the suction pile installation is successful and in agreement with design assumptions, for permanent and mobile moorings, the following data should be monitored and recorded during pile installation:

— distance from intended sea floor location;

— underpressure;

— penetration depth;

— penetration rate;

— verticality;

— orientation.

For permanent mooring systems, other parameters usually monitored include plug stability at all depths and plug heave at final penetration.

A.10.4.4 Other anchor types

A.10.4.4.1 Gravity anchors

No guidance is offered.

A.10.4.4.2 Plate anchors

A.10.4.4.2.1 Geotechnical design

A.10.4.4.2.1.1 Basic considerations

For plate anchors, the ultimate holding capacity is often defined as the ultimate pull-out capacity (UPC), which is the force in the mooring line at which the soil around the anchor fails. At UPC, the plate anchor starts moving through the soil in the general direction of the mooring line with no further increase in resistance or with a gradual reduction in resistance. However, for some designs, the plate anchor starts diving deeper upon overload (i.e., the mooring line force at the anchor is greater than the UPC) until it reaches a more competent soil layer where the overload can be resisted.

The UPC is a function of

— soil undrained shear strength at the anchor fluke,

— projected area of the fluke,

— fluke shape,

— bearing capacity factor, and

— depth of penetration.

When determining the UPC, the disturbance of the soil due to the soil failure mode should be considered. This disturbance is generally accounted for in the form of an empirical reduction factor. This factor should be based on reliable test data and relevant studies [34]. Typically, in order to generate a deep failure mode, the plate anchor's penetration depth in clay should be about 4.5 times the equivalent width of the fluke. If the final depth does not generate a deep failure mode, the bearing capacity factor should be reduced accordingly (see, for example, Reference [34]).

Plate anchors get their high holding capacity from their embedment into more competent soil. Therefore, it is important that the anchor's penetration depth is established during the installation process.

A design method applicable to all types of plate anchors in clay is described in Reference [34]. The various anchor types differ in terms of installation method, but they all end up as a plate anchor after rotation/keying/triggering.

It is important to verify by measurement the installation depth of all types of plate anchor, i.e. that this depth equals the target installation depth of the anchor. During subsequent rotation/keying there is generally a loss in penetration depth which needs to be taken into account when setting the target installation depth.

A plate anchor achieves its UPC by having its fluke oriented nearly perpendicular to the direction of the applied loading. To ensure that the fluke rotates to achieve a maximum projected bearing area (a process called keying/triggering), the plate anchor design and installation procedure should

a) as part of the installation, facilitate rotation of the fluke to a position that ensures that, when the anchor is subjected to a higher tension during a design event, the fluke continues to rotate to a position perpendicular to the direction of the applied tension,

b) ensure no significant loss of penetration occurs during anchor rotation, which can move the fluke into weaker soil. The pullout capacity of plate anchors partly penetrated into a stiffer/harder layer underlying a soft layer should consider the influence of the overlying soft layer on the long-term capacity. Loss of penetration during anchor rotation/keying/triggering in such situations should be given special attention;

c) have the structural integrity to sustain fluke rotation about both horizontal and vertical axes, depending on the type of plate anchor and its installation orientation.

Factors of safety for holding capacity are provided in Table 8 in 10.4.4.

A.10.4.4.2.1.2 Prediction method for drag embedded plate anchor

A.10.4.4.2.1.2.1 General

The following aspects of drag embedded plate anchor performance should be determined:

— anchor line mechanics;

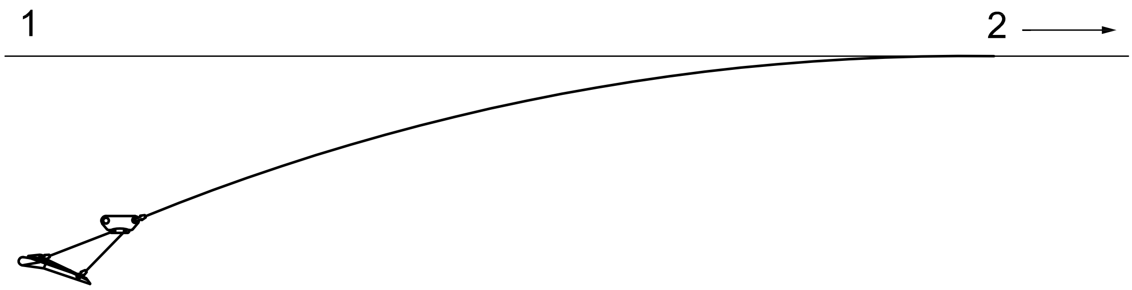
— installation performance;

— holding capacity performance.

All three are closely linked and influence one another, as described in the following sections.

A.10.4.4.2.1.2.2 Anchor line mechanics

As discussed in References [34], [143] and [144], anchor line mechanics strongly influence the drag embedded plate anchor's final orientation and depth below the sea floor, which in turn govern the holding capacity of the anchor system. Figure A.24 is a schematic of an anchor line configuration showing the inverse catenary of the line as it cuts through the soil. As the line tension increases the anchor continues to penetrate and the inverse catenary of the embedded line increases the line angle at the anchor attachment point. The deeper the anchor penetrates, the larger the angle at the anchor attachment point becomes, which ultimately sets the limit for penetration leading to continuous drag of the anchor without further increase in tension.



Key

1 sea floor

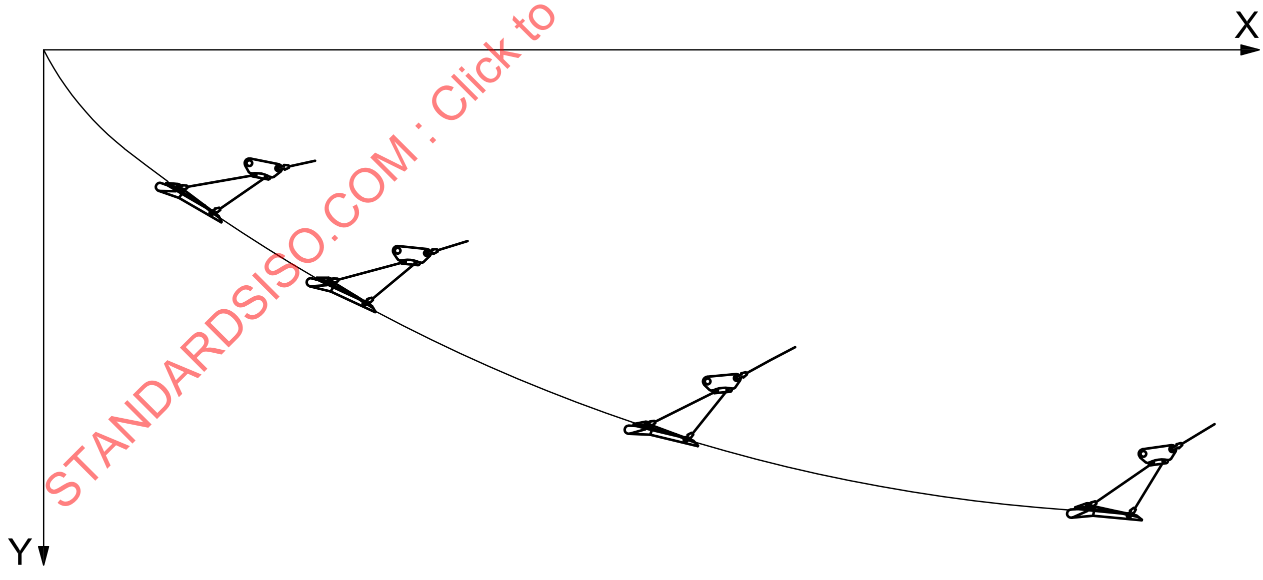
2 mudline force

Figure A.24 — Schematic of anchor line configuration during embedment

In general, this problem is approached in the same manner as that for predicting the displaced shape of a catenary, fixed at both ends, and deformed by its own weight plus bearing pressures exerted from the soil normal to the line and the shear resistance tangential to the line. The governing differential equations for this system of forces are non-linear and require an iterative numerical solution.

A.10.4.4.2.1.2.3 Installation performance

As discussed in References [34], [143] and [144], the capacity of a drag embedded plate anchor depends strongly on its final orientation and depth below the sea floor, hence prediction of the anchor trajectory during installation is a critical issue. Figure A.25 is a schematic diagram showing a typical anchor trajectory and sequence of anchor orientations as the anchor line is dragged along the sea floor.



Key

X drag distance

Y anchor depth

Figure A.25 — Anchor trajectory and fluke orientation during installation

Methods for predicting this trajectory generally fall into the following four groups.

a) Empirical methods: these are typically based on correlations with observed anchor performance and dependent on anchor characteristics (weight) and an approximate measure of the soil resistance. However, many of these field studies are not available in the public domain.

b) Limit equilibrium methods: these take into account a more detailed description of soil and anchor geometry/weight. The methods are based on an estimated soil reaction distribution on the anchor at failure; site specific soil and anchor information can be incorporated in detail. This approach is most commonly used, and commercial software based on this approach is available (see Reference [153]).

c) Plastic limit analysis: this is similar to the Limit equilibrium methods. Virtual work principles are used to minimize the calculated failure capacities with respect to the geometric parameters defining the failure mechanism at any anchor depth, anchor orientation, and anchor line conditions.

d) Advanced numerical methods, including FEM. These have the potential for obtaining a rigorous solution for all aspects of anchor behaviour. In practice, however, they have considerable limitations. A complete solution would require a FEM defining non-linear material behaviour, non-linear boundary conditions, and large strain and large deformation theory. Hence, even a simple anchor trajectory prediction requires substantial effort to formulate, set up, and solve. However, FEM can be used to check calculations or enhance other prediction methods.

A.10.4.4.2.1.2.4 Holding capacity performance

As discussed in References [34], [143] and [144], anchor holding capacity is only a special case of the installation sequence and, hence, the methods underlying installation prediction described above are directly applicable. Provided that the penetration depth of the plate anchor is known and the clay is homogeneous (non-layered) the ultimate holding capacity can be expressed on the basis of conventional bearing capacity theory in conjunction with the anchor line solution:

$$F_{\mathrm{max}} = N_{\mathrm{c}}\, A_{\mathrm{eff}}\, \eta\, s_{\mathrm{u,ave}} \tag{A.28}$$

where

F_{max} is the ultimate holding capacity;

A_{eff} is the effective area of the anchor accounting for shape and projected area;

N_{c} is the bearing capacity factor determined, for example, from the method of characteristics or finite element solutions;

η is the empirical reduction factor accounting for progressive failure (strain-softening) when loaded towards failure; see discussion of this factor in [34];

$s_{\mathrm{u,ave}}$ is the measure of the local average undrained shear strength within the failure zone at the design penetration depth, corrected for the effects of cyclic loading, see Reference [34] for guidance.

For layered clay the ultimate holding capacity will be dependent on the position of the anchor relative to the layer boundary; see Reference [34] for guidance.

Overall, considerable judgment and experience is required to evaluate the input parameters for any of the predictive methods.

An example of an anchor analysis can be found in References [145] and [154].

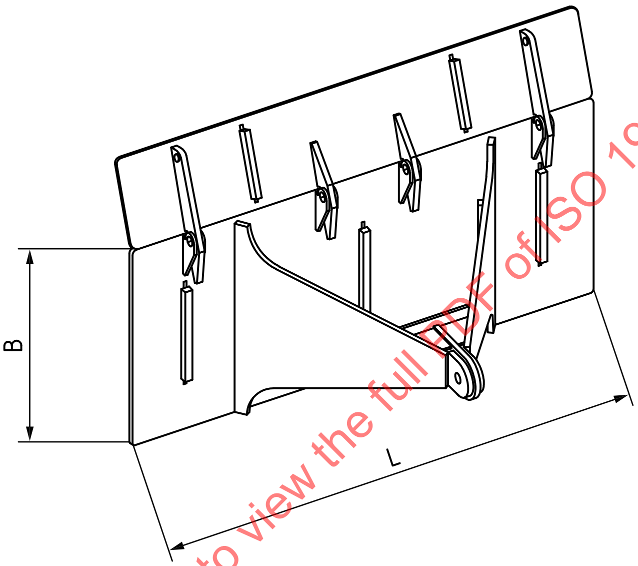
A.10.4.4.2.1.3 Prediction method for direct embedded plate anchors

Anchor capacity determination for direct embedded plate anchors is identical to that shown for drag embedded anchors with the following exceptions:

— final penetration depth is accurately known;

— nominal penetration loss during keying should be included (usually taken as 0,25 to 1,0 times the fluke's vertical dimension, or B in Figure A.26, depending on shank and keying flap configuration);

— calculation of effective fluke area should use an appropriate shape factor and projected area of fluke with keying flap in its set position.



Key

B width

L length

Figure A.26 — Definition of anchor fluke dimensions

A.10.4.4.2.1.4 Factors of safety for drag embedded plate anchors

Factors of safety for drag embedded plate anchors are higher than those for drag anchors because overloading of plate anchors normally results in pull-out, while drag anchors, deeply penetrated in soft to stiff clay, either drag horizontally or dig deeper, thereby developing constant or higher holding capacities (see Table 8 in 10.4.4). For plate anchors that exhibit similar overloading behaviour to drag anchors, consideration can be given to using drag anchor factors of safety, assuming the behaviour can be verified by appropriate field tests and experience.

A.10.4.4.2.2 Installation

A.10.4.4.2.2.1 Direct embedded plate anchors

Direct embedment of plate anchors can be achieved by suction, impact or vibratory hammer, propellant, hydraulic ram, or gravity (see A.11.1.6.5.2).

Installation procedures should be developed and installation analyses should be performed for direct embedded plate anchors to verify that the anchors can be embedded to the design depth. The installation analysis should also consider plate anchor retrieval if applicable.

For the embedment analysis, the risk of causing uplift of the soil plug inside the suction embedment tool should be considered. The allowable underpressure to avoid uplift should exceed the required embedment pressure by a factor of 1,5 (see A.10.4.3.2.2.2.1.4).

Plate anchor installation tolerances should be established and should be considered in the anchor's geotechnical, structural, and installation design. Typical tolerances to be considered are:

— allowable deviation from target heading of the mooring line attachment to limit padeye side loads and rotational moments on the anchor padeye;

— minimum penetration required before keying or test loading to achieve the required holding capacity;

— allowable loss of anchor penetration during plate anchor keying or test loading.

In order to verify that the plate installation is successful and in agreement with the design assumptions, the parameters listed in A.10.4.3.2.4 should also be recorded.

A.10.4.4.2.2.2 Drag embedded plate anchors

For drag embedded plate anchors used in permanent moorings, the anchor design should incorporate adequate installation information to ensure that the anchor reaches the target penetration depth, thereby meeting the safety requirements of the mooring system for the actual soil and design situations. Typical information to be monitored and verified is:

a) drag anchor installation line tension vs. time;

b) catenary shape of installation line based on line tension and line length to verify that uplift at the sea floor during embedment is within allowable ranges and to verify anchor position;

c) direction of anchor embedment;

d) final anchor penetration depth (not dictated by failure of a shear pin).

For further guidance, see Reference [34].

A.10.4.5 Chain and wire rope holding capacity

The sea floor coefficients of friction depend upon the actual ocean bottom at the anchoring location and type of mooring line. Generalized friction factors for chain and wire rope are given in Table A.6. The starting friction factors are normally used to compute the holding capacity of the line and the forces on the line during deployment. In the absence of better data, the coefficients in Table A.5 can be used for various bottom conditions such as soft mud, sand, and clay.

Table A.6 — Sea floor coefficients of friction for chain and wire rope

Line type	Starting	Sliding
Chain	1,00	0,70
Wire rope	0,60	0,25

A.10.4.6 Anchor installation tension

A.10.4.6.1 General

A.10.4.6.1.1 Drag anchors

DNV-RP-E301 [33] provides alternative guidance on how to determine the anchor installation tension.

A.10.4.6.1.2 Suction piles and plate anchors

For suction piles and plate anchors, the installation records should demonstrate that the anchor penetration is within the range of upper and lower bound penetration predictions developed during the anchor geotechnical design. In addition, the installation records should confirm the installation behaviour, i.e. self weight penetration, embedment pressures, drag embedment forces, and that the anchor orientation is consistent with the anchor design. Under these conditions, test loading of the anchor, in accordance with 10.4.6.1 should not be required. However, the mooring and anchor design should define a minimum acceptable level of test loading. This test loading should ensure that the mooring line's inverse catenary is sufficiently formed to prevent unacceptable slacking of the mooring line due to additional change in shape of the embedded part of the line and/or to inverse catenary cut-in during storm conditions.

Plate anchors should be subjected to adequate keying loads to ensure that sufficient anchor fluke rotation takes place and that the associated loss of penetration depth is within that expected and accounted for in the specification of the target penetration depth. The keying tension required and amount of estimated fluke rotation should be based on reliable geotechnical analysis and verified by prototype or scale model testing. The keying analysis used to establish the keying tension should also include analysis of the anchor's rotation when subjected to the ULS intact and redundancy check tensions. If the calculated anchor rotation during keying differs from the anchor rotation during redundancy check conditions, then the anchor's structure should be designed for any resulting out-of-line loading to ensure that the anchor's structural integrity is not compromised.

In cases where the installation records show significant deviation from the predicted values and these deviations indicate that the anchor holding capacity is compromised, test loading of the anchor, as per 10.4.6.1, can be required. This can be an acceptable option to prove holding capacity for temporary moorings. However, testing anchors to the tension used for the ULS intact check does not necessarily prove that required anchor holding safety factors are met, which is of special concern for permanent mooring systems. Consequently, if the installation records show that the anchor holding capacity is significantly smaller than calculated and factors of safety are not met, then the following measures to ensure adequate factors of safety should be considered:

- additional soil investigation at the anchor location to establish and/or confirm soil properties at the anchor site;
- retrieval of the anchor and re-installation at a new undisturbed location;
- retrieval of the anchor, and redesign and reconstruction of the anchor to meet design requirements and re-installation at an undisturbed location;
- delay of vessel hook-up to provide additional soil consolidation.

Drag embedded plate anchors should be test loaded, in accordance with A.10.4.6.1, unless at least one of the following conditions is satisfied:

a) the anchor installation tension (drag-in tension) is equal to or greater than the anchor required test tension (see 10.4.6), and the anchor is not keyed in the opposite direction,

b) soil properties at the anchor location have been established in accordance with A.7.2.3, and the depth of the anchor after keying is known with reasonable accuracy and is equal to or greater than the minimum depth used for the design of the anchor.