INTERNATIONAL STANDARD

ISO 19905-1

Third edition
2023-10

Oil and gas industries including lower carbon energy — Site-specific assessment of mobile offshore units —

Part 1:
Jack-ups: elevated at a site

Industries du pétrole et du gaz, y compris les énergies à faible teneur en carbone — Évaluation spécifique du site d'unités mobiles en mer —

Partie 1: Plateformes auto-élévatrices : Surélévées sur un site

Reference number
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Foreword

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

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

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

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

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

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

This third edition cancels and replaces the second edition (ISO 19905-1:2016), which has been technically revised.

The main changes are as follows:

— updates regarding operations in arctic areas in: Scope, 5.1.4, Figure 5.2-1, 6.7, 7.2, 7.6, 7.8, 10.8, Table 10.3-1 and A.10.8, and added 8.8.9;

— need for Classification revised in Scope and expanded in 5.1.7;

— Clause 3 updated to align with 19900 and other sources. Further definitions added;

— added definitions of symbols for undrained shear strength in 4.1.2;

— added definitions of symbols for horizontal and moment capacity coefficients and cyclic degradation factors in 4.1.5;

— interaction with SSA-I explained in 5.1;

— exposure levels (in 5.5) revised to align with ISO 19900:2019;

— requirements and information on earthquake response analysis gathered in 10.7 and A.10.7 respectively and referenced from 8.6, 8.7, 8.8, A.8.6.3, A.8.7;

— 9.3, A.9.3.1.2, A.9.3.3.1 and A.9.4.1 expanded to include foundation capacities and stiffnesses based on strength parameters rather than applied preload. Clause E.4 added to address the former;

— clarifications of Step 2 foundation checks in 9.3.6;

— 9.4.6 on cyclic mobility expanded to address liquefaction and liquefaction-induced lateral flow and A.9.4.6 expanded accordingly;

— earthquake analysis requirements (in 10.7) revised; reference to 5.5.5 added and text moved from other clauses inserted;

— minor update to alternative analysis methods (see 10.10, formerly 10.9);

— minor clarifications in 13.2;

— clarified that the H_{max} to H_{srp} relationships in A.6.4.2.2 are defaults in the absence of site-specific data; the application of kinematics reduction in A.6.4.2.3 is no longer by means of wave height reduction;

— most probable peak enhancement factor in A.6.4.2.7 now given as a range;

— default current profile in A.6.4.3 revised;

— alternative wind profiles now permitted in A.6.4.6.2;

— added references to ISO 19901-10 and ISO 19901-8 in A.6.5.1.1;

— added reference to liquefaction-induced lateral flows in Table A.6.5-1;

— the requirements for the geotechnical report in A.6.5.1.5.3 have been revised and expanded especially in respect of shear strength;

— penetration in clays in A.9.3.2.2 updated to address strain rate dependency and strain softening;

— squeezing of clay in A.9.3.2.6.2 revised;

— punch-through for sand overlying clay in A.9.3.2.6.4 clarified and formula revised;

— major updates to the ultimate vertical/horizontal/rotational capacity interaction function and parameters in A.9.3.3.2 for spudcans in sand and clay due to the inclusion of further soil profiles in clay and an approach for including the effects of cyclic loading on foundation capacities;

— the effect of cyclic loading on the yield surface has been added in A.9.3.3.7; incorporates text that was in A.9.3.4.2.2;

— revised guidance on the selection of shear modulus for clay in A.9.3.4;

— Step 2a foundation capacity and sliding checks in A.9.3.6.4 revised and the figures corrected;

— guidance on Cyclic mobility in A.9.4.6 significantly expanded, and this clause now also addresses liquefaction and liquefaction-induced lateral flow;

— guidance on structural and foundation modelling expanded in A.10.7.3.2 with particular reference to modelling for earthquake response analysis;

— guidance on ice added in A.10.8;

— guidance in A.12.2.3.2 on non-circular prismatic member classification and in A.12.2.3.3 on reinforced components clarified in respect of slender components;

— sketch in Table 12.3-1 b) corrected;

— clarifications in Table A.12.4-1 and correction to formula in Figure A.12.4-1;

— guidance on strength of tubular members in A.12.5 updated to align with ISO 19902:2020 (combined axial and bending loading in A.12.5.3 of cosine interaction form instead of previous form using linear interaction) and simplified combined axial, bending, beam shear and torsion checks have been added;

— clarified calculation for e in A.12.6.2.3 on axial compressive local strength check;

— clarified F_y in A.12.6.2.5.4 on Class 4 slender-section bending moment strength;

— beam shear area formulations for chord cross sections updated in A.12.6.3.4;

— Table B-2: revised partial resistance factor for horizontal foundation capacity for total stress (clay/undrained) and added partial resistance factors for vertical-horizontal foundation bearing capacity when considering material factored representative soil strength and for calculated foundation capacities.

— corrections to formulae in Figure C.2.4-1, "The drag-inertia method including DAF scaling factor";

— Figure E.1-1 corrected;

— Figure E.3-1 b) corrected;

— added Clause E.4 on calculated foundation capacities approach;

— added Clause E.5 providing an example of a simplified free-field liquefaction assessment calculation method;

— Norway regional requirements in H.2 updated. H.2.2 Regulatory framework and H.2.4 Technical commentary deleted. Added new H.2.3 Technical requirements for jack-up rigs operating close to a permanent occupied installation.

— US Gulf of Mexico requirements (H.3) metocean data replaced by reference to hurricane data from API RP-2MET, 2019. General updates. Unoccupied post-evacuation case expanded.

A list of all parts in the ISO 19905 series can be found on the ISO website.

Any feedback or questions on this document should be directed to the user's national standards body. A complete listing of these bodies can be found at www.iso.org/members.html.

Introduction

This document is one of the International Standards on offshore structures prepared by TC 67/SC 7 (i.e. ISO 19900, the ISO 19901 series, ISO 19902, ISO 19903, ISO 19904-1, the ISO 19905 series and ISO 19906).

NOTE 1 These are sometimes incorrectly referred to as the ISO 19900 series on offshore structures.

The International Standards on offshore structures prepared by TC 67/SC 7 address design requirements and assessments for all offshore structures used by the petroleum and natural gas industries worldwide. Through their application, the intention is to achieve reliability levels appropriate for attended and unattended offshore structures, regardless of the type of structure and the nature or combination of the materials used.

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

The International Standards on offshore structures prepared by TC 67/SC 7 are intended to provide wide latitude in the choice of structural configurations, materials and techniques without hindering innovation. Sound engineering judgment is, therefore, necessary in the use of these documents.

This document, which has been developed from the Society of Naval Architects and Marine Engineers (SNAME) Technical & Research Bulletin 5-5A (2002)[170], states the general principles and basic requirements for the site-specific assessment of mobile jack-ups; it is intended to be used for site-specific assessment and not for jack-up design.

NOTE 2 For the exposure level 1 (L1) assessment and, where appropriate, the exposure level 2 (L2) assessment prior to evacuation being effected, this document requires the use of 50 year independent or 100 year joint probability metocean extremes, together with associated partial action factors. It is based on extensive benchmarking and best practice in the international community.

Site-specific assessment is normally carried out when it is intended to install an existing jack-up unit at a specific site. The assessment is not intended to provide a full evaluation of the jack-up; it assumes that aspects not addressed herein have been addressed using other practices and standards at the design stage. In some instances, the original design of all or part of the structure could be in accordance with other International Standards on offshore structures prepared by TC 67/SC 7, and in some cases, different practices or standards could have been applied.

The purpose of the site assessment is to demonstrate the adequacy of the jack-up and its foundations for the assessment situations and defined limit states, taking into account the consequences of failure. It is important that the results of a site-specific assessment be appropriately recorded and communicated to those persons required to know or act on the conclusions and recommendations. Alternative approaches to the site-specific assessment can be used, provided that they have been shown to give a level of structural reliability equivalent, or superior, to that implicit in this document.

Annex A provides background to and guidance on the use of this document. The clause numbering in Annex A is the same as in the main text in order to facilitate cross-referencing. ISO/TR 19905-2 provides additional background to some clauses and a detailed sample 'go-by' calculation.

NOTE 3 ISO/TR 19905-2:2012 is based on ISO 19905-1:2012. The second ediiton of ISO/TR 19905-2 will be based on this document.

Annex B summarizes the partial factors. Supplementary information is presented in Annexes C to G. Annex H presents regional information.

NOTE 4 The site-specific assessment (SSA) of a jack-up normally comprises the two parts: an elevated SSA (SSA-E), addressed in this document, and an installation and removal SSA (SSA-I), which is planned to be addressed in an International Standard as part of the ISO 19905 series.

In this document, the following verbal forms are used:

— “shall” indicates a requirement;

— “should” indicates a recommendation;

— “can” indicates a possibility or a capability;

— “may” indicates a permission.

Oil and gas industries including lower carbon energy — Site-specific assessment of mobile offshore units —

Part 1: Jack-ups: elevated at a site

1 Scope

This document specifies requirements and provides recommendation and guidance for the elevated site-specific assessment (SSA-E) of independent leg jack-up units for use in the petroleum and natural gas industries. It addresses:

a) occupied non-evacuated, occupied evacuated and unoccupied jack-ups;

b) the installed (or elevated) phase at a specific site.

It also addresses the requirement that the as-installed condition matches the assumptions used in the assessment.

This document does not address the site-specific assessment of installation and removal (SSA-I).

To ensure acceptable reliability, the provisions of this document form an integrated approach, which is used in its entirety for the site-specific assessment of a jack-up.

When assessing a jack-up operating in regions subject to sea ice and icebergs, it is intended that the assessor supplements the provisions of this document with the relevant provisions relating to ice actions contained in ISO 19906 and procedures for ice management contained in ISO 35104. This document does not address design, transit to and from site, or installation and removal from site.

This document is applicable only to independent leg mobile jack-up units that are structurally sound and adequately maintained, which is normally demonstrated through holding a valid recognized classification society, classification certificate. Jack-ups that do not hold a valid recognized classification society certificate are assessed according to the provisions of ISO 19902, supplemented by methodologies from this document, where applicable.

NOTE 1 Well conductors can be a safety-critical element for jack-up operations. However, the integrity of well conductors is not part of the site-specific assessment process for jack-ups and is, therefore, not addressed in this document. See A.1 for guidance on this topic.

NOTE 2 RCS rules and the IMO MODU code (International Maritime Organisation Mobile Offshore Drilling Unit code) provide guidance for the design of jack-ups.

2 Normative references

The following documents are referred to in the text in such a way that some or all of their content constitutes requirements of this document. For dated references, only the edition cited applies. For undated references, the latest edition of the referenced document (including any amendments) applies.

ISO 19900:2019, *Petroleum and natural gas industries — General requirements for offshore structures*

ISO 19901-1:2015, *Petroleum and natural gas industries — Specific requirements for offshore structures — Part 1: Metocean design and operating conditions*

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

ISO 19901-4, *Petroleum and natural gas industries — Specific requirements for offshore structures — Part 4: Geotechnical and foundation design considerations.*

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

ISO 19906:2019, *Petroleum and natural gas industries — Arctic offshore structures*

ISO 35104, *Petroleum and natural gas industries — Arctic operations — Ice management*

ISO 35106, *Petroleum and natural gas industries — Arctic operations — Metocean, ice, and seabed data*

3 Terms and definitions

For the purposes of this document, the terms and definitions given in ISO 19901-2, ISO 19901-4, ISO 19906 and the following apply.

ISO and IEC maintain terminology databases for use in standardization at the following addresses:

— ISO Online browsing platform: available at https://www.iso.org/obp

— IEC Electropedia: available at https://www.electropedia.org/

3.1
abnormal environmental event
environmental *hazardous event* (3.31) having probability of occurrence not greater than 10^{-3} per annum (1 in 1 000 years)

[SOURCE: ISO 19900:2019, 3.1]

3.2
abnormal wave crest
wave crest with probability of typically 10^{-3} to 10^{-4} per annum

3.3
accidental event
non-environmental *hazardous event* (3.31) having probability of occurrence not greater than 10^{-3} per annum (1 in 1 000 years)

Note 1 to entry: Accidental events, as referred to in this document, are associated with a substantial release of energy, such as vessel collisions, fires, and explosions.

Note 2 to entry: Lesser accidents that could be expected during the life of the structure, such as dropped objects and low energy vessel impact, are termed incidents and are addressed under operational design situations.

[SOURCE: ISO 19900:2019, 3.2]

3.4
action
external load applied to the *jack-up* (3.36) (direct action) or an imposed deformation or acceleration (indirect action)

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

[SOURCE: ISO 19900:2019, 3.3, modified — "structure" changed to "jack-up".]

3.5
action effect
result of *actions* (3.4) on a *structural member* (3.87) or *structural component* (3.86) (e.g. internal force, moment, stress, strain) or on the *jack-up* (3.36) (e.g. deflection, rotation)

[SOURCE: ISO 19900:2019, 3.4, modified — "structural member" added and "structure" changed to "jack-up".]

3.6
assessment
site-specific assessment
evaluation of the stability and structural integrity of a *jack-up* (3.36) and, where applicable, its seabed restraint or support against the actions determined in accordance with specific requirements

Note 1 to entry: The specific requirements are given in this document.

Note 2 to entry: An assessment can be limited to an evaluation of the components or members of the structure which, when removed or damaged, could cause failure of the whole structure, or a significant part of it.

3.7
assessment criteria
quantitative formulations describing the conditions to be fulfilled for each *assessment situation* (3.9)

[SOURCE: ISO 19900:2019, 3.15, modified — References to "design" deleted.]

3.8
assessment resistance
resistance limit calculated using factored *representative values* (3.64) of *basic variables* (3.13) or from factored expressions based on unfactored *representative values* (3.64) of *basic variables* (3.13)

EXAMPLE Examples of basic variables relevant to resistance are material properties.

[SOURCE: ISO 19900:2019, 3.12, modified — "design" changed to "assessment".]

3.9
assessment situation
set of physical conditions for which the *jack-up* (3.36) or its components are verified

Note 1 to entry: For discussion on configuration, see 5.4.1.

Note 2 to entry: The assessment situations are checked against the acceptance criteria of this document to demonstrate that the relevant limit states are not exceeded.

[SOURCE: ISO 19900:2019, 3.16, modified — Reference to "design" deleted and "structure" changed to " jack-up".]

3.10
assessor
entity performing the site-specific assessment

3.11
backfill
submerged weight of all of the soil that can be present on top of the spudcan

Note 1 to entry: Backfilling can occur during or after preloading. $W_{BF,o}$ refers to the submerged weight of the backfilling that occurs up to achieving the preload reaction. $W_{BF,A}$ refers to the submerged weight of the backfilling that occurs after the maximum preload has been applied and held. Both $W_{BF,o}$ and $W_{BF,A}$ can comprise backflow and/or infill. For discussion of the effects, see A.9.3.2.1.4.

3.12
backflow
soil that flows from beneath the spudcan around the sides and onto the top

Note 1 to entry: Backflow is part of *backfill* (3.11).

3.13
basic variable
variable representing physical quantities which characterize *actions* (3.4) and environmental influences, geometric quantities, or material properties including soil properties

Note 1 to entry: Basic variables are typically uncertain random variables or random processes used in the calculation or assessment of representative values of actions or resistance.

[SOURCE: ISO 19900:2019, 3.7]

3.14
boundary conditions
actions and/or constraints on a structural member (or a group of structural members) by other structural members or by the surrounding environment

Note 1 to entry: Boundary conditions can be used to generate reaction forces at locations of restraint.

3.15
characteristic value
value assigned to a *basic variable* (3.13) with a prescribed probability

Note 1 to entry: In some design/assessment situations, a variable can have two characteristic values, an upper value and a lower value.

[SOURCE: ISO 19900:2019, 3.9]

3.16
chart datum
CD
local datum used to fix water depths on a chart or tidal heights over an area

Note 1 to entry: Chart datum is usually an approximation to the level of the lowest astronomical tide.

[SOURCE: ISO 19901-1:2015, 3.2]

3.17
consequence category
classification system for identifying the environmental, economic and indirect personnel safety consequences of failure of a platform used to determine *exposure level* (3.21)

[SOURCE: ISO 19902:2020, 3.11]

3.18
dynamic amplification factor
DAF
ratio of a dynamic action effect to the corresponding static action effect

Note 1 to entry: For a jack-up, the dynamic action effect is best simulated by means of a concentrated or distributed inertial loadset. It is usually not appropriate to factor the static actions to simulate the effects of dynamic actions.

Note 2 to entry: In this document the DAFs used are either $K_{DAF,SDOF}$ for a single degree of freedom analogy or $K_{DAF,RANDOM}$, for a stochastic simulation, see 4.1.1.

3.19
deterministic analysis
analysis in which the response is determined from a single combination of actions

3.20
earthquake response spectrum
function representing the peak elastic response for single degree of freedom oscillators with specific damping ratios in terms of absolute acceleration, pseudo velocity, or relative displacement values against natural frequency or period of the oscillators

[SOURCE: ISO 19901-2:2022, 3.13, modified — "earthquake" added to term.]

3.21
exposure level
classification system used to establish relevant criteria for a *jack-up* (3.36) based on consequences of failure

Note 1 to entry: An exposure level 1 (L1) jack-up is the most critical and exposure level 3 (L3) the least (see ISO 19900:2019, 7.3).

[SOURCE: ISO 19900:2019, 3.20, modified — "structure" replaced with "jack-up" and Note to entry added.]

3.22
extreme storm event
extreme combination of wind, wave and current conditions used for the assessment of the *jack-up* (3.36)

Note 1 to entry: This is the metocean event used for ULS storm assessment (see 5.5.4 and 6.4).

3.23
field
general area where the *jack-up* (3.36) is intended to operate

Note 1 to entry: The field is a general area as opposed to the *site* (3.74) which is specific.

3.24
fixed load
permanent parts of the *jack-up* (3.36), including hull, legs and spudcans, outfit, stationary and moveable-fixed equipment

Note 1 to entry: Moveable-fixed equipment normally includes the drilling package structure and associated permanently attached equipment.

3.25
footprint
sea floor depression that remains when a *jack-up* (3.36) is removed from a site

3.26
foundation
soil and spudcan supporting a *jack-up* (3.36) leg

3.27
foundation fixity
rotational restraint offered by the soil to the spudcan

3.28
foundation stability
ability of the foundation to provide sufficient support to remain stable when subjected to actions and incremental deformation

3.29
global analysis
determination of a consistent set of internal forces and moments, or stresses, in a structure that are in equilibrium with a defined set of actions on the entire structure

Note 1 to entry: When a global analysis is of a transient situation (e.g. earthquake), the inertial response is part of the equilibrium.

3.30
hazard
potential source of harm

Note 1 to entry: Harm is typically differentiated between harm to people, harm to the environment, or harm in terms of costs to organization(s) or society in general.

[SOURCE: ISO 19900:2019, 3.26]

3.31
hazardous event
event that occurs when a *hazard* (3.30) interacts with a *jack-up* (3.36)

EXAMPLE Wave or iceberg impacting the *jack-up*, excessive weight added to the *jack-up*, vessel collision and scour in the vicinity of the *jack-up*.

[SOURCE: ISO 19900:2019, 3.27, modified — "structure" changed to "jack-up", Example modified to include iceberg and to exclude fire, explosion, and landslip.]

3.32
independent leg jack-up
jack-up unit with legs that can be raised and lowered independently

3.33
inertial loadset
set of actions that approximates the effect of the inertial forces

Note 1 to entry: An inertial loadset is used only in quasi-static analyses.

3.34
infill
soil above the plan area of the spudcan arising from sediment transport or hole sidewall collapse

Note 1 to entry: Infill is part of *backfill* (3.11).

3.35
intrinsic wave frequency
wave frequency of a periodic wave in a reference frame that is stationary with respect to the wave

Note 1 to entry: If there is no current, the reference frame is also stationary with respect to the sea floor. If there is a current, the reference frame moves with the same speed and in the same direction as the current.

3.36
jack-up
mobile offshore unit with a buoyant hull and one or more legs that can be moved up and down relative to the hull

Note 1 to entry: A jack-up reaches its operational mode by lowering the leg(s) to the seabed and then raising the hull to the required elevation. The majority of jack-ups have three or more legs, each of which can be moved independently and which are supported in the seabed by spudcans.

3.37
jack-up owner
owner
representative of the company or companies owning or chartering the jack-up

Note 1 to entry: The energy company, the *operator* (3.53), contracts the jack-up and is generally not the owner or charterer.

3.38
joint probability metocean data
combinations of wind, wave and current that produce the action effect that can be expected to be exceeded at a site, on average, once in the return period

3.39
leaning instability
instability of an independent leg jack-up that can arise when the rate of increase of actions on the foundation with jack-up inclination exceeds the rate of increase of foundation capacity with depth

3.40
limit state
state beyond which the *jack-up* (3.36) or a *structural member* (3.87) no longer satisfies the assessment criteria

3.41
load case
compatible load arrangements, sets of deformations and imperfections considered simultaneously with permanent actions and fixed variable actions for a particular design or verification

3.42
long-term operation
operation of a jack-up on one particular site for more than the recognised classification society special survey period

3.43
lowest astronomical tide
LAT
level of low tide when all harmonic components causing the tides are in phase

Note 1 to entry: The harmonic components are in phase approximately once every 19 years, but these conditions are approached several times each year.

[SOURCE: ISO 19901-1:2015, 3.17]

3.44
mat-supported jack-up
jack-up unit with the leg(s) rigidly connected by a foundation structure, such that the leg(s) are raised and lowered in unison

3.45
mean high water spring tidal level
arithmetic mean of all high water spring tidal sea levels measured over a long period, ideally 19 years

3.46
mean low water spring tidal level
arithmetic mean of all low water spring tidal sea levels measured over a long period, ideally 19 years

3.47
mean sea level
MSL
arithmetic mean of all sea levels measured at hourly intervals over a long period

Note 1 to entry: Seasonal changes in mean level can be expected in some regions and over many years the mean sea level can change.

[SOURCE: ISO 19901-1:2015, 3.20]

3.48
mean zero-upcrossing period
average intrinsic period of the zero-upcrossing waves in a sea state

Note 1 to entry: In practice, the mean zero-crossing period is often estimated from the zeroth and second moments of the wave spectrum as given by Formula (3.41-1):

$$T_z = T_2 = \sqrt{m_0(f)/m_2(f)} = 2\pi\sqrt{m_0(\omega)/m_2(\omega)} \quad (3.41\text{-}1)$$

where

f is the frequency in cycles per second (Hertz);

m_0 is the zeroth spectral moment and is equivalent to σ^2, the variance of the corresponding time series;

m_2 is the second spectral moment;

T_2 and T_z are the average zero-upcrossing period of the water surface elevation, defined by the zeroth and second order spectral moments, ($T_2 = T_z$);

ω is the wave frequency in radians per second.

[SOURCE: ISO 19901-1:2015, 3.22, modified — "intrinsic" deleted, "(up or down) zero-crossing" changed to "upcrossing" and definitions of terms in the equation added]

3.49
most probable maximum extreme
MPME
value of the maximum of a variable with the highest probability of occurring over a defined period of time

Note 1 to entry: A defined period of time can be, for example, X hours.

Note 2 to entry: The most probable maximum extreme is the value for which the probability density function of the maxima of the variable has its peak. It is also called the mode or modus of the statistical distribution.

[SOURCE: ISO 19901-1:2015, 3.24, modified — Added "over a defined period of time" and note 1 to entry.]

3.50
nominal strength
strength calculated for a cross-sectional area, taking into account the stress raising effects of the macro-geometrical shape of the component of which the section forms a part, but disregarding the local stress raising effects from the section shape and any weldment or other fixing detail

3.51
nominal value
value assigned to a variable specified or determined on a non-statistical basis, typically from acquired experience or physical conditions, or as published in a recognized code or standard

Note 1 to entry: In some design/assessment situations, a variable can have two nominal values, an upper value

3.52
operating manual
marine operations manual
latest approved document that defines the operational characteristics and capabilities of the jack-up

Note 1 to entry: The *assessor* (3.10) is should ensure that any updated weight data are provided.

3.53
operator
representative of the company or companies leasing the site

Note 1 to entry: The operator is normally the oil company acting on behalf of co-licensees.

[SOURCE: ISO 19900:2019, 3.35, modified — Note 2 to entry deleted.]

3.54
performance
ability of a *jack-up* (3.36) or a *structural member* (3.87) and the foundation to fulfil specified requirements

Note 1 to entry: Specified requirements include requirements for structural and foundation integrity and functionality.

[SOURCE: ISO 19900:2019, 3.36 modified — Added "and the foundation", and "structure" replaced with "jack-up" and " structural component" replaced with "structural member".]

3.55
preloading
installation and embedment of the spudcans by vertical loading of the soil beneath a jack-up leg spudcan with the objective of ensuring sufficient foundation capacity under assessment situations through to the time when the maximum load is applied and held

Note 1 to entry: While three-legged jack-ups preload by taking water ballast on board, jack-ups with four or more legs typically achieve foundation preload by carrying the hull weight on pairs of legs in turn. This procedure is known as pre-driving and generally does not require the addition of water ballast. For the purposes of this document, no distinction is made between preload and pre-drive.

3.56
preload reaction
maximum vertical reaction under a spudcan supporting the in-water weight of the jack-up during the entire preloading operation

Note 1 to entry: The in-water weight is the full weight of the hull, variable load and preload ballast, plus the legs and spudcans and any contained water, reduced by the buoyancy in water of the legs and spudcans (calculated from their external dimensions). Soil buoyancy and the weight of any soil backfill above the spudcan are neglected. It is necessary to take care when accounting for water contained in the spudcan (in some cases this can be included in the quoted leg weight).

Note 2 to entry: This is the maximum reaction on a spudcan which would be obtained during preloading if the jack-up were installed on an infinitely rigid foundation.

Note 3 to entry: The preload reaction is a key parameter in the geotechnical analysis of independent leg foundations. Assessors consider values that can be reasonably achieved during preload operations. The assessment is invalidated if the value considered in the site assessment is not achieved during preload operations.

3.57
punch-through
rapid, uncontrolled vertical leg movement due to soil failure in strong soil overlying weak soil

3.58
quasi-static
static representation of a dynamic process

Note 1 to entry: In some cases, the influence of structural accelerations can be approximated by using an equivalent inertial loadset.

3.59
rack phase difference
RPD
relative difference in the position of adjacent leg chords within a leg measured parallel to the longitudinal axis of the chords

Note 1 to entry: This is the out-of-plane distortion of the plan-frame.

3.60
recognized classification society
RCS
member of the international association of classification societies (IACS), with recognized and relevant competence and experience in jack-ups, and with established rules and procedures for classification/certification of such units used in petroleum-related activities

[SOURCE: ISO 19901-7:2013, 3.23, modified — "floating structures" replaced by "jack-ups", "installations" replaced by "such units".]

3.61
redundancy
ability of a structure to find alternative load paths following structural failure of one or more components, thus limiting the consequences of such failures

Note 1 to entry: Statically determinate structures, contrary to statically indeterminate structures, do not generally exhibit redundancy.

[SOURCE: ISO 19902:2020, 3.38]

3.62
regulator
authority established by a national governmental administration to oversee the activities of the offshore oil and natural gas industries within its jurisdiction, with respect to the overall safety to life and protection of the environment

Note 1 to entry: The term "regulator" can encompass more than one agency in any particular territorial waters.

Note 2 to entry: The regulator can appoint other agencies, such as marine classification societies, to act on its behalf, and in such cases, regulator as it is used in this document includes such agencies.

Note 3 to entry: In this document, the term "regulator" does not include any agency responsible for approvals to extract hydrocarbons, unless such agency also has responsibility for safety and environmental protection.

[SOURCE: ISO 19902:2020, 3.39]

3.63
reliability
performance (3.54) over a specified period of time

Note 1 to entry: When reliability is used in the context of limit states, it can be expressed as the probability that the limit is not exceeded.

Note 2 to entry: The specified period of time is typically one year.

[SOURCE: ISO 19900:2019, 3.39]

3.64
representative value
value assigned to a *basic variable* (3.13) for verification of a *limit state* (3.40) in an *assessment situation* (3.9)

Note 1 to entry: Two types of representative value used in verification are *characteristic value* (3.15) and *nominal value* (3.51).

[SOURCE: ISO 19900:2019, 3.40, modified — "Design" deleted.]

3.65
resistance
ability to withstand *action effects* (3.5)

Note 1 to entry: When undertaking an assessment, the resistance checks normally include: Overturning stability, foundation, holding system, *structural members* (3.83) and *structural components* (3.82).

3.66
return period
average period between occurrences of an event

Note 1 to entry: The offshore industry commonly uses a return period measured in years for environmental events. The return period in years is equal to the reciprocal of the annual probability of occurrence of the event.

Note 2 to entry: For the purpose of this definition, events include both discrete hazardous events as well as exceedances of a threshold value of a relevant variable.

[SOURCE: ISO 19900:2019, 3.42]

3.67
scatter diagram
joint probability of two or more (metocean) parameters

Note 1 to entry: A scatter diagram is especially used with wave parameters in the metocean context, see ISO 19901-1:2015, A.5.8. The wave scatter diagram is commonly understood to be the probability of the joint occurrence of the significant wave height (H_s) and a representative period ($T_{z,i}$ or $T_{p,i}$).

[SOURCE: ISO 19901-1:2015, 3.29, modified — In Note to entry, "(for example in fatigue assessments)" replaced by ", see ISO 19901-1:2015, A.5.8".]

3.68
scour
removal of seabed material from the foundation due to current and waves

3.69
sea floor
interface between the sea and the *seabed* (3.71)

[SOURCE: ISO 19900:2019, 3.46]

3.70
sea state
condition of the sea during a period in which its statistics remain approximately stationary

Note 1 to entry: In a statistical sense the sea state does not change markedly within the period. The period during which this condition exists is usually assumed to be three hours, although it depends on the particular weather situation at any given time.

[SOURCE: ISO 19901-1:2015, 3.31]

3.71
seabed
materials below the *sea floor* (3.69)

[SOURCE: ISO 19900:2019, 3.47]

3.72
shallow gas
gas pockets or entrapped gas below impermeable layers at shallow depth

3.73
significant wave height
statistical measure of the height of waves in a sea state

Note 1 to entry: The significant wave height was originally defined as the mean height of the highest one-third of the mean zero upcrossing waves in a sea state. In most offshore data acquisition systems, the significant wave height is currently taken as $4\sqrt{m_0}$ (where m_0 is the zeroth spectral moment, see ISO 19901-1:2015, 3.37) or 4σ, where σ is the standard deviation of the time series of water surface elevation over the duration of the measurement, typically a period of approximately 30 min.

[SOURCE: ISO 19901-1:2015, 3.35]

3.74
site
specific position and orientation at which a *jack-up* (3.36) operates within a *field* (3.23)

3.75
skirt
vertical bulkhead(s), closed in plan view, beneath the main body of a *spudcan* (3.81)

3.76
skirted spudcan
spudcan (3.81) with a *skirt* (3.75)

3.77
sliding
horizontal movement of a spudcan

3.78
special survey
extensive and complete survey carried out at each nominal year interval, which closes a cycle of annual classification and mandatory surveys

Note 1 to entry: This is also referred to as "renewal survey" by some IACS members. The special survey period is normally between five and eight years.

3.79
spectral density function
spectrum
energy density function
measure of the variance associated with a time-varying variable per unit frequency band and per unit directional sector

Note 1 to entry: Spectrum is a shorthand expression for the full and formal name of spectral density function or energy density function.

Note 2 to entry: The spectrum is, in general, written with two arguments: one for the frequency variable and one for a direction variable.

Note 3 to entry: Within ISO 19901-1, the concept of a spectrum applies to waves, wind turbulence and action effects (responses) that are caused by waves or wind turbulence. For waves, the spectrum is a measure of the energy traversing a given space.

Note 4 to entry: Not to be confused with an earthquake response spectrum.

[SOURCE: ISO 19901-1:2015, 3.39, modified — Deleted first sentence of Note 2 to entry. Added Note 4 to entry.]

3.80
spectral peak period
period of the maximum (peak) energy density in the *spectrum* (3.79)

Note 1 to entry: In practice, there is often more than one peak in a spectrum.

Note 2 to entry: There are two types of spectral peak period used within this document: intrinsic and apparent. The distinction is discussed in A.7.3.3.5, which is, in turn, based on ISO 19901-1:2015, 8.4.4 and A.8.4.3.

[SOURCE: ISO 19901-1:2015, 3.38, modified — Added Note 2 to entry.]

3.81
spudcan
structure at the base of a leg supported by the soil

3.82
squeezing
lateral movement of weak soil between the spudcan base and an underlying stronger layer, or of weak soil between two stronger layers

3.83
stochastic analysis
analysis in which a probabilistic approach is taken to model the random nature of the variables of interest

Note 1 to entry: In general, a linear(ized) stochastic analysis can be performed in the frequency domain or in the time domain, whereas non-linear stochastic analysis can only use time domain simulations. This document does not support frequency domain stochastic analysis.

3.84
stress concentration factor
SCF
factor relating a local stress to the nominal stress at a detail

[SOURCE: ISO 19902:2020, 3.49, modified — Note 1 to entry deleted.]

3.85
structural analysis
process or algorithm for determining action effects from a given set of actions

Note 1 to entry: Structural analyses are performed at three levels [global analysis of an entire structure, analysis of part of a structure (e.g. a leg), local analysis of a structural member and local analysis of a structural component] using different structural models.

[SOURCE: ISO 19902:2020, 3.50, modified — Note 1 to entry with added example and reference to "structural member".]

3.86
structural component
component
physically distinguishable part of a member cross-section of uniform yield strength

Note 1 to entry: The cross-section of a non-tubular member is usually comprised of several structural components. A component consists of only one material. Where a plate component is reinforced by another piece of plating, the reinforcement can be of a different yield strength. See also further discussion in A.12.1.1.

3.87
structural member
member
physically distinguishable part of a braced structure connecting two joints

Note 1 to entry: A structural member can also be defined as the leg of a non-truss leg jack-up.

Note 2 to entry: See also further discussion in A.12.1.1.

3.88
sudden hurricane
sudden cyclone
sudden typhoon
sudden tropical revolving storm that forms locally and, due to speed of formation and proximity to infrastructure at time of formation, might not allow sufficient time to evacuate occupied facilities within the time required by the emergency evacuation plan

Note 1 to entry: The intent is that the jack-up be assessed to L1 for the specified sudden tropical revolving storm, see 5.5.2 and 5.5.3.

3.89
sustained wind speed
time-averaged wind speed with a defined averaging duration of 1 min or longer at a specified elevation

[SOURCE: ISO 19901-1:2015, 3.43, modified — Duration changed from "10 min or longer".]

3.90
undrained shear strength
maximum shear stress at yielding or at a specified maximum strain in an undrained condition

Note 1 to entry: Yielding is the condition of a material in which a large plastic strain occurs at little or no stress increase.

Note 2 to entry: Strain softening is also to be considered.

[SOURCE: ISO 19901-8:2023, 3.42, modified — Added Note 2 to entry.]

3.91
utilization
member utilization
foundation utilization
maximum absolute value of the ratio of the generalized representation of the assessment action effect to the generalized representation of the assessment resistance in compatible units

Note 1 to entry: Utilizations are calculated for each limit state of the assessment situation being considered.

Note 2 to entry: Only utilizations smaller than or equal to 1,0 satisfy the assessment criteria for a particular limit state.

Note 3 to entry: The assessment action effect is the response to the factored actions. The assessment resistance is the representative resistance divided by the partial resistance factor.

Note 4 to entry: For members and foundations subjected to combined forces, the internal force pattern and the resistance combine into an interaction formula. If the interaction formula governing the assessment check is, or can be, reduced to an inequality of the form $U \leq 1{,}0$, then the utilization is equal to U.

3.92
variable load
items carried by the jack-up to support its operation that are not included in the fixed load

3.93
water depth
vertical distance between the sea floor and still water level

Note 1 to entry: As there are several options for the still water level (see A.6.4.4), there can be several water depth values. Generally, assessment water depth is determined to the extreme still water level.

Note 2 to entry: The water depth used for calculating wave kinematics varies between the maximum water depth of the mean high water spring tide plus a positive storm surge, and the minimum water depth of the mean low water spring tide less a negative storm surge, where applicable.

[SOURCE: ISO 19901-1:2015, 3.47, modified — Notes to entry rewritten.]

4 Symbols and abbreviated terms

4.1 Symbols

4.1.1 General

A_{F_d}	action effect due to factored actions
B_S	soil buoyancy of spudcan below bearing area, i.e. the submerged weight of soil displaced by the spudcan below D_embed, the greatest embedment depth of maximum cross-sectional spudcan bearing area below the sea floor
C_mr	moment reduction factor

D_{embed} greatest embedment depth of maximum cross-sectional spudcan bearing area below the sea floor

D_e equivalent set of inertial actions representing dynamic extreme storm effects or ground motion effects due to earthquakes

E_e metocean actions due to the extreme storm event

f_{FD} fatigue damage design factor

F_d assessment load case (see 8.8)

F_H horizontal force applied to the spudcan due to the assessment load case (see 8.8)

F_V gross vertical force acting on the soil beneath the spudcan due to the assessment load case F_d (see 8.8)

G_F actions due to the fixed load positioned such as to adequately represent their vertical and horizontal distribution

G_v actions due to maximum or minimum variable load, as appropriate, positioned at the most onerous centre of gravity location applicable to the configurations under consideration

K effective length factor

$K_{DAF,RANDOM}$ DAF from random wave time domain (stochastic) analyses, including the mean values, obtained from a random wave calculation. It is the ratio of the absolute value of a dynamic action effect to the absolute value of the corresponding static action effect, each including their mean value

$K_{DAF,SDOF}$ DAF from single degree-of-freedom representation of dynamic behaviour, excluding the mean values, obtained from a single degree-of-freedom (SDOF) calculation. It is the ratio of the amplitude of a dynamic action effect to the amplitude of the corresponding static action effect for periodic excitation of a linear one degree-of-freedom model approximation of jack-up behaviour

L_1 length of the vector from a specified origin to the action effect

L_2 length of the vector from the origin specified for L_1 to the factored interaction surface

L_{b1} length of the vector from origin used for establishing the bearing utilization $(F_H, F_V)_{ORG}$ to the environmental response point (determined from the factored actions) (F_H, F_V)

L_{b2} length of the vector from origin used for establishing the bearing utilization $(F_H, F_V)_{ORG}$ and passing through (F_H, F_V) to the factored vertical-horizontal capacity surface $Q_{VH,f}$

L_{s1} length of the vector from origin used for establishing the sliding utilization $(F_H, F_V)_{ORG}$ to the environmental response point (determined from the factored actions) (F_H, F_V)

L_{s2} length of the vector from origin used for establishing the sliding utilization $(F_H, F_V)_{ORG}$ and passing through (F_H, F_V) to the factored vertical-horizontal capacity surface $Q_{VH,fv}$

L_{AE} length of the vector from a specified origin to the action effect

L_{IS} length of the vector from the same origin to the factored interaction surface

M_{OTM} overturning moment due to factored actions

N number of cycles to failure in fatigue of a specified constant amplitude stress range

Q_H maximum horizontal foundation capacity

R factored resistance

$R_{d,OTM}$ factored stabilizing moment

$R_{r,OTM}$ representative stabilizing moment

T_n jack-up natural period

T_p apparent modal or peak period of the wave spectrum

$T_{p,i}$ intrinsic modal or peak period of the wave spectrum

$T_{z,i}$ intrinsic mean zero-upcrossing period of the water surface elevation in a sea state

U utilization

$U_{S,pl}$ utilization of preload

$U_{S,sl}$ utilization of foundation resistance to sliding

$U_{S,vhm}$ utilization of vertical and horizontal foundation capacity

V_{Lo} maximum vertical reaction under the spudcan considered required to support the in-water weight of the jack-up during the entire preloading operation (this is not the soil capacity; see 3.56)

V_{st} vertical force applied to the spudcan due to the assessment load case (see 8.8) (includes effects of leg weight and water buoyancy but excludes effects of backfill and spudcan soil buoyancy)

$W_{BF,A}$ submerged weight of the backfill that occurs after the maximum preload has been applied and held

$W_{BF,o}$ submerged weight of the overburden on top of the spudcan from backfill during preloading

$\gamma_{f,D}$ partial action factor applied to the inertial actions due to dynamic response

$\gamma_{f,E}$ partial action factor applied to the metocean or earthquake actions

$\gamma_{f,G}$ partial action factor applied to the actions due to fixed load

$\gamma_{f,V}$ partial action factor applied to the actions due to the variable load

$\gamma_{R,H}$ partial resistance factor for holding system strength

$\gamma_{R,Hfc}$ partial resistance factor for horizontal foundation capacity

$\gamma_{R,OTM}$ partial resistance factor for stabilizing moment

$\gamma_{R,PRE}$ partial resistance factor for preload

$\gamma_{R,S}$ partial resistance factor for spudcan strength

γ_{VH} partial resistance factor for foundation capacity

4.1.2 Symbols used in A.6

D_1 directional spreading function as a function of n

D_2	directional spreading function as a function of s
D_3	directional spreading function as a function of σ
d_w	water depth
$F(\alpha_w)$	directionality function
f	wave frequency
H_{max}	individual extreme wave height
H_s	increased significant wave height to account for wave asymmetry
H_{srp}	significant wave height for the assessment return period
h_{ref}	reference depth for wind-driven current
L_w	wave length of the wave with H_{max} and T_{ass} in water depth d_w, according to the periodic wave theory used
N	inverse exponent of the power law wind profile
n	parameter exponent in D_1
S_y	smallest spacing between the legs of 3-legged jack-ups
$S_{PM}(\omega)$	Pierson-Moskowitz wave spectrum for a sea state
$S_{JS}(\omega)$	JONSWAP wave spectrum for a sea state
$S_{\eta\eta}(f)$	wave spectral density function expressed as a function of wave frequency
$S_{\eta\eta}(f,\alpha_w)$	directional short-crested power density spectrum
s	parameter in D_2
$s_{u,ave}$	average undrained shear strength = $(s_{uC} + s_{uD} + s_{uE})/3$
s_{uC}	static triaxial compression undrained shear strength
s_{uD}	static DSS undrained shear strength
s_{uE}	static triaxial extension undrained shear strength
T_{ass}	intrinsic wave period associated with H_{max}
T_p	apparent modal or peak period of the wave spectrum
$T_{p,i}$	intrinsic modal or peak period of the wave spectrum
$T_{z,i}$	intrinsic mean zero-upcrossing period of the water surface elevation in a sea state
V_C	current velocity as a function of z
V_s	downwind component of associated surge current (excluding wind-driven component)
V_{ref}	1 min sustained wind speed at elevation Z_{ref} (normally at 10 m above MSL)
V_t	downwind component of mean spring tidal current

V_w wind generated surface current

V_Z the wind speed at elevation Z above SWL under consideration

Z elevation above SWL under consideration

z vertical coordinate relative to SWL under consideration, positive upwards

Z_{ref} reference elevation above MSL

α_w angle between the direction of elementary wave trains and the dominant direction of the short-crested waves

γ shape parameter of the peak enhancement factor in the JONSWAP spectrum

κ kinematics reduction factor

ϕ directional spreading factor based on latitude

σ standard deviation of the normal distribution in D_3

Ψ latitude

4.1.3 Symbols used in A.7

A_{cs} cross-sectional area of member

A_e equivalent area of leg per unit height

A_i equivalent area of member or gusset i

A_{Wi} projected area of the block i perpendicular to the wind direction

C_A added mass coefficient

C_{De} equivalent value of the drag coefficient of a leg bay

C_{Dei} equivalent value of the drag coefficient of member i

C_D, C_{Di} drag coefficient, drag coefficient of member i

$C_{Dpr}(\theta)$ drag coefficient related to the projected diameter

C_{D0} drag coefficient for a tubular with appropriate roughness

C_{D1} drag coefficient for flow normal to the rack related to projected diameter, W

C_m, C_{mi} inertia coefficient, inertia coefficient of member i

C_{me} equivalent value of the inertia coefficient of a leg bay

C_{mei} equivalent value of the inertia coefficient of member i

C_s shape coefficient

D_r, D_i reference diameter, reference diameter of member i

D_e equivalent diameter of leg

D_F face width of leg, outside dimensions, orthogonal to the flow direction

$D_{pr}(\theta)$ projected diameter

d_w water depth

H_s increased significant wave height to account for wave asymmetry

L_w wave length

l_i length of member i node to node centre

m_a added mass contribution (per unit length) for a member

P_i pressure at the centre of block i

s height of one bay, or part of bay considered

T_i intrinsic period of a periodic wave (in a reference frame that is stationary with respect to the wave, i.e. with no current present)

T_n first natural period of surge or sway motion of the jack-up

T_p apparent modal or peak period of the wave spectrum

$T_{p,i}$ intrinsic modal or peak period of th wave e spectrum

T_z apparent mean zero-upcrossing period of the water surface elevation in a sea state

$T_{z,i}$ intrinsic mean zero-upcrossing period of the water surface elevation in a sea state

t_m marine growth thickness

W projected width

$\dot{r}_n$ velocity of the considered member, normal to the member axis and in the direction of the combined particle velocity

$\ddot{r}_n$ acceleration of the considered member, normal to the member axis and in the direction of the combined particle velocity

u wave particle velocity

u_n wave particle velocity resolved normal to the member axis

$\dot{u}_n$ wave particle acceleration resolved normal to the member axis

V_C current velocity for use in the hydrodynamic model

V_f far field (undisturbed) current velocity

V_{zi} wind velocity at the centre of block i

v_n fluid particle velocity resolved normal to the member axis

z' modified coordinate for use in particle velocity formulation

z vertical coordinate relative to SWL under consideration, positive upwards

α_i angle between flow direction and member axis projected onto a horizontal plane

β_i angle defining the member inclination from horizontal

ΔF wave action per unit length

ΔF_{drag} drag action per unit length

$\Delta F_{inertia}$ inertia action per unit length

ρ_a mass density of air

ρ_w mass density of water

θ angle in degrees

ζ_w instantaneous water level (same axis system as z)

4.1.4 Symbols used in A.8

A_{eq} axial area of equivalent leg model beam

A_{seff} effective shear area of the equivalent leg model beam

E Young's modulus of steel

F applied axial action

G shear modulus

I second moment of area

K_{hh} horizontal leg-to-hull connection stiffness

K_{rh} rotational leg-to-hull connection stiffness

K_{vh} vertical leg-to-hull connection stiffness

L_c cantilevered length (from the hull to the seabed reaction point)

L_{ub} distance from the spudcan reaction point to the hull vertical centre of gravity

M applied moment

P applied shear

P_g sum of the leg forces due to functional actions on legs at hull, including the weight of the legs above the hull

Δ axial deflection (shortening) of the leg at the point of force application from the detailed leg model

Δ_C axial end displacements of the combined leg and leg-to-hull connection model

Δ_M lateral deflection of the cantilevered leg at the point of moment application from the detailed leg model

δ_C lateral deflection of the cantilevered leg at the point of moment application from the combined leg and leg-to-hull connection model

δ_M lateral deflection of the cantilevered leg at the point of moment application from the detailed leg model

θ_C slope of the end of the cantilever from the combined leg and leg-to-hull connection model

θ_M slope of the cantilever at the point of moment application from the detailed leg model

θ_P slope of the cantilever at the point of shear application from the detailed leg model

4.1.5 Symbols used in A.9 and Annex E

A spudcan effective bearing area based on cross-section taken at uppermost part of bearing area in contact with soil (see Figure A.9.3-3)

A_s spudcan laterally projected embedded area

a depth interpolation parameter

a_s bearing capacity squeezing factor constant

B effective spudcan diameter at uppermost part of bearing area in contact with the soil (for rectangular footing B equal to width)

B_{max} diameter of the contact area in plan when the spudcan is fully seated

B_S soil buoyancy of spudcan below bearing area i.e. the submerged weight of soil displaced by the spudcan below D_{embed}, the greatest embedment depth of maximum cross-sectional spudcan bearing area below the sea floor

b_s bearing capacity squeezing factor constant dependent on spudcan diameter

C_H horizontal capacity coefficient

$C_{H,NC}$ horizontal capacity coefficient for the normally consolidated case per A.9.3.3.2 a) i)

$C_{H,U}$ horizontal capacity coefficient for the uniform strength case per A.9.3.3.2 a) ii)

$C_{H,shallow}$ horizontal capacity coefficient at shallow embedment

$C_{H,deep}$ horizontal capacity coefficient at deep embedment

C_M moment capacity coefficient

$C_{M,NC}$ moment capacity coefficient for the normally consolidated case per A.9.3.3.2 a) i)

$C_{M,U}$ moment capacity coefficient for the uniform strength case per A.9.3.3.2 a) ii)

D_{embed} greatest embedment depth of maximum cross-sectional spudcan bearing area below the sea floor (see Figure A.9.3-3)

D_b depth of backflow; infill should not be considered

D_R relative density of sand (percent)

d depth beneath sea floor

d_c bearing capacity depth factor

d_{crit} depth at which maximum bearing resistance occurs (layered case)

d_q depth factor on surcharge for drained soils

d_γ depth factor on self weight for drained soils

$f_{cy,V}$	cyclic degradation factor on vertical capacity
$f_{cy,H}$	cyclic degradation factor on horizontal capacity
$f_{cy,M}$	cyclic degradation factor on moment capacity
F_H	horizontal force applied to the spudcan due to the assessment load case (see 8.8)
F_M	moment force applied to the spudcan due to the assessment load case (see 8.8)
F_V	gross vertical force acting on the soil beneath the spudcan due to the assessment load case F_d (see 8.8)
$(F_V/Q_V)_t$	vertical load at intersection of adhesion yield surface and foundation yield surface
f_1	factor applied to horizontal capacity used in yield surface formula for embedded spudcans on clay
f_2	factor applied to moment capacity used in yield surface formula for embedded spudcans on clay
f_r	foundation rotational stiffness reduction factor
G_{max}	maximum value of the shear modulus (of the foundation soil) which occurs at small strain
H	distance from spudcan maximum bearing area to weaker layer below
H_{cav}	limiting depth of cavity that remains open above the spudcan during penetration
h_1	embedment depth to the uppermost part of the spudcan, (if not fully embedded, $h_1 = 0$)
h_2	spudcan tip embedment depth
I_{rNC}	rigidity index for normally consolidated clays
I_P	plasticity index
j	dimensionless stiffness factor
k_a	active earth pressure coefficient (for $s_u = 0$)
k_p	passive earth pressure coefficient
K_1, K_2, K_3	stiffness factors for vertical, horizontal and rotational foundation stiffness respectively
K_{d1}, K_{d2}, K_{d3}	depth factors for vertical, horizontal and rotational foundation stiffness respectively
K_s	coefficient of punching shear
L_s	length of strip footing
m	parameter to define effect of adhesion on the foundation yield surface envelope
n_s	load spread factor for sand overlying clay
N_c	bearing capacity factor, taken as $N_c\, s_c = 6{,}0$ for circular footings
N_q	bearing capacity factor for a flat rough circular footing

N_γ	bearing capacity factor for a flat rough circular footing
p_o'	effective overburden pressure at greatest embedment depth, D_{embed}, of maximum bearing area
p_a	atmospheric pressure
Q_0	spudcan bearing capacity at sea floor
Q_H	maximum horizontal foundation capacity
Q_{Hs}	foundation sliding capacity
Q_M	ultimate moment capacity of foundation
Q_{Mp}	increased ultimate moment capacity due to further spudcan penetration under environmental actions
Q_{Mps}	ultimate moment capacity when further spudcan penetration leads to full contact of the entire underside of the spudcan with the seabed
Q_{Mpv}	ultimate moment capacity under further spudcan penetration, when the applied vertical force is too low to achieve full contact of the entire underside of the spudcan with the seabed
Q_{peak}	maximum bearing capacity at $d = d_{crit}$
$Q_{u,b}$	ultimate vertical foundation bearing capacity assuming the spudcan bears on the surface of the lower (bottom) clay layer with no backfill
Q_V	gross ultimate vertical foundation capacity
Q_{Vnet}	net ultimate vertical foundation capacity
Q_{Vo}	initial gross ultimate vertical foundation capacity established by preload operations
r_f	failure ratio
R_{OC}	over-consolidation ratio
s_c	bearing capacity shape factor
s_u	undrained shear strength
$s_{u,a}$	undrained shear strength of backfill material above the spudcan
s_{uo}	undrained shear strength at greatest embedment depth of maximum bearing area, D_{embed}, below sea floor
s_{uH}	undrained shear strength at depth of H_{cav} below sea floor
$s_{u,l}$	undrained shear strength at the spudcan tip
s_{um}	undrained shear strength at the sea floor
$s_{u,b}$	undrained shear strength of lower clay layer below spudcan
$s_{u,t}$	undrained shear strength of upper clay layer below spudcan

$s_{u,ned}$ minimum undrained shear strength near (within ¼ spudcan diameter below) the embedment depth

$s_{u,nml}$ minimum undrained shear strength within ¼ spudcan diameter below the mudline.

T thickness of weak clay layer underneath spudcan

V_D volume of the spudcan below the maximum bearing area that is penetrated into the soil

V_L available spudcan reaction

V_{Lo} maximum vertical reaction under the spudcan considered required to support the in-water weight of the jack-up during the entire preloading operation (this is not the soil capacity; see 3.56)

V_{st} vertical force applied to the spudcan by the assessment load case, see 8.8, (includes effects of leg weight and water buoyancy but excludes effects of backfill and spudcan soil buoyancy)

V_{spud} the total volume of the spudcan beneath the backfill

V_{sw} gross vertical spudcan reaction under still water conditions for the spudcan being considered (includes effects of backfill and spudcan soil buoyancy)

W_{BF} submerged weight of the backfill

$W_{BF,A}$ submerged weight of the backfill that occurs after the maximum preload has been applied and held

$W_{BF,o}$ submerged weight of the overburden on top of the spudcan from backfill during preloading

$W_{BF,omin}$ minimum value of the submerged weight of the backfill, due to backflow during preloading

α_s adhesion factor

β equivalent cone angle

δ steel/soil friction angle in degrees

$\gamma_{R, Hfc}$ partial resistance factor for horizontal foundation capacity

$\gamma_{R, VH}$ partial resistance factor for foundation capacity

γ' submerged (effective) unit weight of soil

ρ_{su} rate of increase in undrained shear strength with depth

ϕ' effective angle of internal friction for sand in degrees

ν Poisson's ratio

4.1.6 Symbols used in A.10

B effective spudcan diameter at uppermost part of bearing area in contact with the soil

C_{rd} radiation damping coefficient of a dashpot (force per unit velocity)

D_e	equivalent set of inertial actions representing dynamic extreme storm effects or ground motion effects due to earthquakes
E_e	metocean actions due to the extreme storm event
$F_{BS,Amplitude}$	single amplitude of quasi-static base shear over one wave cycle
$F_{BS,(QS)Max}$	maximum quasi-static wave/current base shear
$F_{BS,(QS)Min}$	minimum quasi-static wave/current base shear
F_{in}	magnitude of the inertial loadset
G	shear modulus
G_F	actions due to the fixed load positioned such as to adequately represent their vertical and horizontal distribution
G_o	shear modulus of the foundation soil
G_v	actions due to maximum or minimum variable load, as appropriate, positioned at the most onerous centre of gravity location applicable to the configurations under consideration
$K_{DAF,RANDOM}$	DAF from random wave time domain (stochastic) analyses
$K_{DAF,SDOF}$	DAF from single degree-of-freedom representation of dynamic behaviour
K_{eff}	effective system stiffness
M_{eff}	effective system mass
O_T	total horizontal offset of the leg base with respect to the hull
O_1	offset due to leg-to-hull clearances
O_2	offset due to maximum hull inclination permitted by the operating manual
T_n	first natural period of surge or sway motion of the jack-up
T_p	apparent modal or peak period of the wave spectrum
$T_{p,i}$	intrinsic modal or peak period of the wave spectrum
ν	Poisson's ratio (of the foundation soil)
Ω	ratio of jack-up natural period to wave excitation period
ρ	total, saturated, (mass) density of the foundation soil
ζ	damping ratio or fraction of critical damping
ζ_{rd}	radiation modal damping ratio to account for spudcan vertical motion
ω_n	natural frequency (rad/s)

4.1.7 Symbols used in A.11

$D_{c,e}$	calculated existing fatigue damage prior to arriving at site

$D_{c,s}$ calculated fatigue damage during planned operations on site

$f_{FD,e}$ fatigue damage design factor applicable to $D_{c,e}$

$f_{FD,s}$ fatigue damage design factor applicable to $D_{c,s}$

N number of cycles to failure in fatigue of a specified constant amplitude stress range, S

S constant amplitude stress range

4.1.8 Symbols used in A.12

A gross cross-sectional area

A_{ec} total effective area of a slender section in compression of a non-circular prismatic member

A_c cross-sectional area for use in the assessment of a non-circular prismatic member in compression

A_{eff} effective area of a plate with reinforcement

$A_{eff,i}$ effective area of a component i of a non-circular prismatic member in compression

A_f cross-sectional area of a semi-compact section of a non-circular prismatic member

A_i cross-sectional area of the ith component comprising the structural member

A_o the area enclosed by the median line of the perimeter material of a section

A_p fully plastic effective cross-sectional area of a non-circular prismatic member

A_t cross-sectional area for use in the assessment of a non-circular prismatic member in tension

A_v effective shear area of a non-circular prismatic member in the direction being considered

B_{maf} member moment amplification factor for the axis under consideration

B_s overall breadth of cross-section

b_w width of the wall of a component forming the closed perimeter of a section

b effective width of a component

b_1 width of base plate

b_2 width of reinforcing plate

C_{mr} moment reduction factor

C_x critical elastic buckling coefficient

D outside diameter of a tubular

D_s overall depth of cross-section

$d_{w,lim}$ limiting equivalent head of water

d_i distance between the centroid of the ith component and the plastic neutral axis

E Young's modulus of steel (elastic modulus)

e eccentricity between the axis used for structural analysis and that used for structural strength checks

e_a effective eccentricity between the axis used for structural analysis and that used for structural strength checks for class 3 members

F_{cr} reduced material strength

F_y yield strength in stress units

F_{yeff} effective yield strength of the cross-section of a non-circular prismatic member in stress units

F_{yi} yield strength of the *i*th component of the cross-section of a non-circular prismatic member in stress units

F_{ymin} minimum yield strength of all components in the cross-section of a non-circular prismatic member (minimum value of F_{yi}, in stress units)

$F_{y,ltb}$ yield strength, F_y of the material that first yields when bending about the minor axis

g acceleration due to gravity

h subscript referring to the component that produces the smallest value of P_{pl}

I second moment of area

I_e effective second moment of area of a non-circular prismatic member cross-section

I_f second moment of area of a plastic, a compact or a semi-compact section of a non-circular prismatic member cross-section

I_{pt} polar moment of inertia of a tubular

I_{pp} polar moment of inertia a non-circular prismatic member

I_1 major axis second moment of area of the gross cross-section

I_2 minor axis second moment of area of the gross cross-section

J torsion constant

K effective length factor

L_b effective length of a beam-column between supports

L_p limiting plastic length

L_r limiting unbraced length for inelastic torsional bucking

L_{ub} unbraced length of member for the plane of flexural buckling

M_b representative bending moment strength of a tubular or a non-circular prismatic member

M_{by}, M_{bz} representative bending moment strength about member y- and z-axes, respectively

M_p plastic moment strength of a tubular or a non-circular prismatic member

M_{py}, M_{pz} plastic moment strengths of a tubular or a non-circular prismatic member about member y- and z-axes, respectively

M_u bending moment in a member due to factored actions determined in an analysis that includes global P-Δ effects

M_{ua} amplified bending moment determined from M_{ue}

M_{ue} corrected effective bending moment determined from M_u

M_{uay}, M_{uaz} amplified bending moments due to factored actions about member y- and z-axes, respectively

M_{uey}, M_{uez} corrected bending moments due to factored actions about member y- and z-axes, respectively

M_{uy}, M_{uz} bending moments due to factored actions about member y- and z-axes, respectively, determined in an analysis that includes global P-Δ effects

P_a representative axial compressive strength of a tubular

P_E Euler buckling capacity

P_n representative axial compressive strength based on local strength for column buckling of a non-circular prismatic member

P_p representative axial strength of a non-circular prismatic member

P_{pl} representative local axial compressive strength of non-circular prismatic member prismatic members

P_t representative axial tensile strength of a non-circular prismatic member

P_u axial force in a member due to factored actions determined in an analysis that includes global P-Δ effects

P_{ut} axial tensile force due to factored actions

P_{uc} axial compressive force due to factored actions

P_v representative shear strength of a tubular

P_{vy}, P_{vz} are the representative shear strengths in the local y- and z-directions of a non-circular prismatic member, respectively

P_{xe} representative elastic local buckling strength of a tubular

P_y plastic strength of a non-circular prismatic member

P_{yc} representative local buckling strength of a tubular

p depth below sea floor (zero if above sea floor)

r_{ltb} radius of gyration about the minor axis when used for lateral-torsional buckling considerations

r radius of gyration for the plane of flexural bending

r_t maximum distance from centroid to an extreme fibre for torsional shear check

S_e reduced effective section modulus of a slender section of a non-circular prismatic member

S_f elastic section modulus of a semi-compact section of a non-circular prismatic member

S_y, S_z section moduli for use in the assessment of a non-circular prismatic member in flexure

T_u torsional moment due to factored actions

T_v representative torsional strength of a tubular

t wall thickness of a tubular

t_1 thickness of base plate

t_2 thickness of reinforcing plate

t_f thickness of a flange component

t_w thickness of a web component

V beam shear due to factored actions

V_y, V_z beam shears due to factored actions in the local y- and z-directions, respectively

y_i distance from the neutral axis associated with I_e to the critical point i

Z_p fully plastic (effective) section modulus

α factor that varies depending on the applied loading

γ' submerged (effective) unit weight of soil

$\gamma_{R,Pa}$ partial resistance factor for axial strength of a non-circular prismatic member

$\gamma_{R,Pb}$ partial resistance factor for bending strength of a non-circular prismatic member

$\gamma_{R,Pcl}$ partial resistance factor for local axial compressive strength of a non-circular prismatic member

$\gamma_{R,Pt}$ partial resistance factor for axial tensile strength of a non-circular prismatic member

$\gamma_{R,Pc}$ partial resistance factor for axial compressive strength of a non-circular prismatic member

$\gamma_{R,Pv}$ partial resistance factor for torsional and beam shear strength of a non-circular prismatic member

$\gamma_{R,Tb}$ partial resistance factor for bending strength of a tubular

$\gamma_{R,Tt}$ partial resistance factor for axial tensile strength of a tubular

$\gamma_{R,Tc}$	partial resistance factor for axial compressive strength of a tubular
$\gamma_{R,Tv}$	partial resistance factor for torsional and beam shear strength of a tubular
k	buckling coefficient
λ	column slenderness parameter
λ_h	ratio b/t or $2R/t$ as applicable for component h
λ_c	prismatic column slenderness parameter for a non-circular prismatic member
λ_r	elastic plate slenderness parameter
λ_p	plastic plate slenderness parameter
λ_{plim}	limiting plate slenderness ratio
λ_{po}	plate slenderness ratio coefficient
η	exponent for biaxial bending, a constant dependent on the prismatic member cross-section geometry
ρ	reduction coefficient
ρ_w	mass density of water
σ_1	compressive stress if σ_2 tensile or the larger compressive stress if σ_2 is also compressive
σ_2	tensile stress if σ_2 tensile or the smaller compressive stress if σ_2 is compressive
ψ	ratio of compression to bending stress

4.2 Abbreviated terms

ALE	abnormal-level earthquake
ALS	abnormal/accidental limit state
BS	base shear
BSTF	base shear transfer function
CD	chart datum
DAF	dynamic amplification factor
ELE	extreme level earthquake
FE	finite element
FLS	fatigue limit state
IACS	International Association of Classification Societies
LAT	lowest astronomical tide
LRFD	load and resistance factor design
LTB	lateral torsional buckling
MPM	most probable maximum

MPME	most probable maximum extreme
MSL	mean sea level
OCR	over-consolidation ratio
PDF	probability density function
PSIIP	project specific in-service inspection programme
RCS	recognized classification society
ROV	remotely operated vehicle
RPD	rack phase difference
SCF	stress concentration factor
SDOF	single degree-of-freedom
SLS	serviceability limit state
SSA-E	site-specific assessment for the elevated condition
SSA-I	site-specific assessment for installation and removal
SWL	still water level
TRS	tropical revolving storm
ULS	ultimate limit state
VIV	vortex induced vibration

5 Overall considerations

5.1 General

5.1.1 Interaction with SSA-I

The site-specific assessment of a jack-up normally comprises the two parts, an SSA-E, adressed in this document, and an SSA-I. While different personnel can carry out these assessments, much of the same information is used in both, including:

- jack-up data;
- geotechnical data;
- geophysical information.

Conversely, there is other data that is only used in an SSA-E or only used in an SSA-I. For example, the metocean extremes are used in an SSA-E only whereas the normally expected metocean conditions and propensity for squalls are used in an SSA-I only.

Complicating the issue is that much of the data used for the SSA-I is most efficiently obtained at the same time as the data collected for the SSA-E (e.g. soil data). Failure to collect all the data required for both analyses at the same time can be costly and inefficient.

The normal expectation is that the SSA-E will be undertaken before the SSA-I. However, there can be cases in which this order is reversed, e.g. when the SSA-E is likely to produce a favourable conclusion but emplacing the jack-up could be problematic.

There can be cases in which the results of the SSA-E can affect how the jack-up is to be emplaced on site. Some examples include:

— required heading;

— required preloading level;

— proximity to adjacent structures;

— limits on cantilever extension;

— site remediation e.g. gravel bags, etc.

These limitations should be passed on to those undertaking the SSA-I.

5.1.2 Competency

Assessments undertaken in accordance with this document shall be performed only by persons competent through education, training and experience in the relevant disciplines.

5.1.3 Planning

Adequate planning shall be undertaken before a site-specific assessment is started. The planning shall include the determination of all assessment situations relevant for the site under consideration. The assessment criteria shall be in accordance with Clause 13.

5.1.4 Assessment situations and associated criteria

The assessment situations shall include both extreme events and operational modes because the critical mode of operation is not always obvious. The assessor shall use site-specific metocean, earthquake and geotechnical data, as applicable, for the assessment. The assessment situations and associated criteria are jointly specified in the remainder of this document. They form one whole and shall not be separated from one another.

For mobile offshore drilling units operating in regions subject to sea ice and icebergs, the requirements of this document shall be supplemented with the relevant provisions relating to ice actions contained in ISO 19906 and procedures for ice management contained in ISO 35104. See 10.8 and A.10.8.

NOTE In some cases, ice actions can be mitigated by an ice management plan and/or seasonal operations.

5.1.5 Reporting

The assessor should prepare a report summarizing the inputs, assumptions and conclusions of the assessment. A recommended contents list is given in Annex G.

5.1.6 Regulations

Each country can have its own set of regulations concerning offshore operations. It is the responsibility of the operator and jack-up owner to identify the applicable rules and regulations, depending upon the site and type of operations to be conducted.

5.1.7 Classification of unit

This document is applicable to independent leg jack-ups that are structurally sound and adequately maintained. To achieve this, the unit shall either:

— hold a valid classification society certification from an RCS throughout the duration of the operation at the specific site subject to assessment; or

— have been verified by an independent competent body to be structurally fit for purpose for elevated situations and are subject to periodic inspection, both to the standards of an RCS.

Jack-ups that do not conform with this requirement shall be assessed in accordance with the provisions of ISO 19902, supplemented by methodologies from this document, where applicable.

5.2 Assessment approach

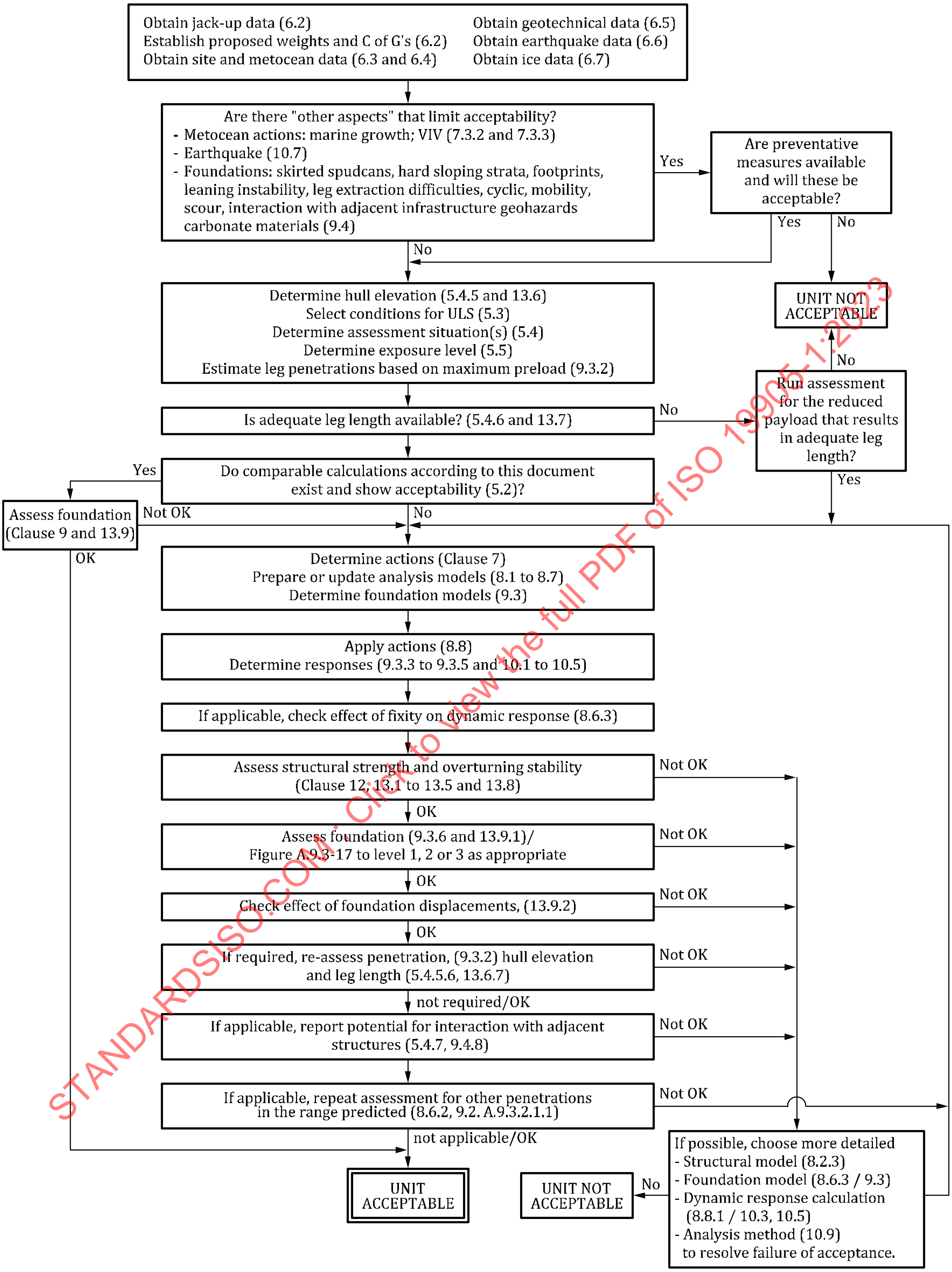
This subclause provides an overview of the data required, the assessment methodology, and the acceptance criteria. An example of a flow chart for extreme storm assessment is shown in Figure 5.2-1. Annex A provides additional information and guidance, including detailed calculation methodology. Annex B specifies the partial factors for use in the assessment. Annexes C to F provide supplementary information or alternative calculation methodologies. Annex G provides a recommended contents list for the assessment report. Annex H provides regional information and provision for Norway and US Gulf of Mexico. ISO/TR 19905-2 provides background to some of the recommendations given in this document and a detailed sample calculation. Other assessment methodologies may be applied, provided that they have been shown to give a level of structural reliability equivalent, or superior, to that implicit in this document.

The assessment of the jack-up can be carried out at various levels of complexity as expanded in a), b) and c) (in order of increasing complexity). The objective of the assessment is to show that the acceptance criteria of Clause 13 are met. If this is achieved at a certain complexity level, there is no requirement to consider a higher complexity level. In all cases, the adequacy of the foundation shall be assessed to level b) or c).

a) Compare assessment situations with design conditions or other existing assessments determined in accordance with this document.

b) Carry out appropriate calculations in accordance with the simpler methods (e.g. pinned foundation, SDOF dynamics) given in this document. Where possible, compare results with those from existing more detailed/complex (e.g. secant or yield interaction foundation model, time domain dynamics) calculations.

c) Carry out appropriate detailed calculations in accordance with the more complex methods (e.g. secant, yield interaction or continuum foundation model, time domain dynamics) given in this document.



NOTE 1 A cross-referenced clause number includes reference to the corresponding clause in Annex A.

NOTE 2 This figure does not fully address: Long term applications (Clause 11); Temperature (13.10); Earthquake (6.6, 7.7, 8.8.8, 10.7)

Figure 5.2-1 — Flow chart for the overall extreme storm assessment

5.3 Selection of assessment situations

ISO 19900 divides the assessment situations into four categories as described in this subclause.

a) Operational and Extreme assessment situations.

The site-specific assessment shall include evaluation of extreme assessment situations with combinations of extreme level metocean actions and the associated storm mode gravity actions. Earthquake and ice actions shall also be considered in combination with the associated permanent and operational gravity actions; however, evaluation is required only in some areas of the world. The applicable partial action and resistance factors for the extreme assessment situation and exposure level shall be as summarized in Annex B. For the associated Ultimate Limit State (ULS), the integrity of the structure should be unimpaired, but damage to the non safety-critical (secondary) structure of the jack-up can be tolerated.

Extreme assessment situations shall be assessed with the jack-up in the most critical operating configuration (increased variable load, cantilever extended and unequal leg loads) when the extreme level metocean conditions are

— within the defined serviceability limits for the jack-up (i.e. the metocean conditions are less severe than those defined for changing to the elevated storm configuration), or

— severe weather occurs with insufficient warning for the unit to be put in to storm configuration, e.g. squalls.

Consideration of the operating configuration is particularly important when the factored functional actions are close to the preload reaction and a small additional leg reaction due to metocean actions can cause significant additional penetration.

Operational assessment situations use the operational metocean conditions with the associated operating mode gravity actions and configuration. For jack-ups where the operations manual permits increases in, or redistribution of, the variable load with reduced metocean conditions (operating configuration, nomograms, etc.), the assessor shall establish an operational assessment situation. Where nomograms are used, a representative selection of situations applicable to the site shall be assessed (e.g. the extreme storm event and one or more less severe metocean conditions).

NOTE The situations above are often found in benign areas where the extreme level metocean conditions are within the defined serviceability limits for the jack-up and do not exceed the limits for changing the jack-up to the elevated storm configuration.

b) Serviceability assessment situations

Serviceability assessment situations are normally covered by the limits specified in the operations manual and, therefore, it is not necessary to assess it unless the operational configuration requirements for the site are outside those limits. However, the requirements of a) above always apply.

c) Fatigue assessment situations

The FLS is generally addressed at the design stage. It is not necessary to evaluate fatigue unless the jack-up is to be deployed for a long-term operation (see Clause 11).

d) Abnormal/Accidental assessment situations

Accidental assessment situations, addressing abnormal environmental events or accidental events, are generally addressed at the design stage and it is not necessary to evaluate them in the assessment unless there are unusual risks at the site under consideration. Abnormal situations shall be assessed when necessary e.g. abnormal level earthquake (ALE) or abnormal level ice assessment (ALS).

5.4 Determination of assessment situations

5.4.1 General

A jack-up can be used in various modes at a single site (e.g. drilling mode/workover mode/tender mode/production mode). In each mode, the jack-up can be in the operating or storm survival configuration. Where more than one configuration is contemplated, the differences (e.g. the varying hull elevations required for each, skidding the cantilever in for a storm, reducing variable deck load) shall be considered in the assessment. The practicality of any required configuration change shall be evaluated and appropriate assumptions incorporated into the assessment calculations. Any required restrictions on the operations shall be included in the operating procedures. The assessment situations shall be determined from appropriate combinations of mode, configuration and limit state.

Where the assessment indicates that an assessment situation does not meet the acceptance criteria of Clause 13, the assessment configuration may be adjusted to achieve acceptability, providing that any resulting deviations from the standard operating procedure of the jack-up are practically achievable, are documented and are communicated by the jack-up owner to their offshore personnel and, if relevant, to the operator. Alternatively, metocean data applicable to the season(s) of operation may be considered.

5.4.2 Reaction point and foundation fixity

The assumed reaction point at the spudcan shall be documented in the assessment report. The jack-up's legs are normally assumed to be pinned at the reaction point. Any divergence from this assumption shall be stated.

NOTE The assumption of pinned footings is a conservative approach for the bending moment in the leg in way of the leg-to-hull connection; see 8.6.3.

5.4.3 Extreme storm event approach angle

The critical extreme storm event approach angles relative to the jack-up are usually different for the various checks that shall be made (e.g. strength versus overturning checks). The critical direction for each check shall be used.

5.4.4 Weights and centre of gravity

For each limit state and configuration being assessed, the appropriate magnitude and position of the fixed and variable loads shall be used. The tolerances on both magnitude and position shall be considered when determining the weights and centres of gravity to use in the assessment.

Where the location of the cantilever, substructure, etc., or the hull elevation, differ between the elevated operating and storm survival configuration, the practicality of making the changes required to achieve the storm survival configuration shall be established.

5.4.5 Hull elevation

The hull elevation used in the assessment shall conform with the requirements specified in 13.6. Generally, this is the larger of that required to maintain adequate clearance with

— adjacent structures, such as a fixed platform, and

— the wave crest.

5.4.6 Leg length reserve

The assessor shall determine the necessity for a reserve of leg length above the upper guides to account for any uncertainty in the prediction of penetration and to provide a contingency against settlement or scour. Leg length reserve requirements are given in 13.7.

5.4.7 Adjacent structures

The potential interaction of the jack-up with any adjacent structures shall be reported, as appropriate. Aspects requiring consideration by the operator include the effects of potential contact with adjacent infrastructure (jacket, subsea structure, pipeline, etc.), the effects of the jack-up's spudcans on the foundation of the adjacent structure and the effects of relative motions on well casing, drilling equipment and well surface equipment (risers, connectors, flanges, etc.).

5.4.8 Other

The assessment is based on the best available information on the conditions at the site. In some cases, it can be found that the actual conditions at the site are inconsistent with the information used, e.g. penetration, eccentricity of spudcan support, orientation, leg inclination. In other cases, the effects of factors such as large guide clearances and sensitivity to RPD cannot be properly quantified prior to installation. In all such cases, the validity of the assessment shall be confirmed once the jack-up has been installed.

NOTE The RPD is usually a good indicator of the degree of eccentricity and the acceptability of the resulting action effects when elevated.

5.5 Exposure levels

5.5.1 Determination of exposure level

Jack-ups can be categorized by various levels of exposure to determine criteria that are appropriate for their intended service and the assessment situations. The exposure levels are determined by consideration of life-safety and of environmental and economic consequences as described in ISO 19900:2019, 7.3.

5.5.2 Exposure level L1

Occupied, non-evacuated jack-ups and jack-ups with high environmental consequence shall be classified to the most onerous exposure level, L1, for all assessment situations.

For extreme storm assessments L1 jack-ups shall be assessed either for the 50 year independent extremes with partial action factor of 1,15 or for the 100 year joint probability metocean data with partial action factor of 1,25 (see 8.8.1 and Annex B).

NOTE 1 The equivalency of the alternatives was justified for temperate climates.

NOTE 2 During the TRS season in TRS areas, it can be appropriate to also assess to the ALS for abnormal storm conditions. For example the 2 500 year return period full population is typically used in the Gulf of Mexico.

5.5.3 Exposure level L2

Occupied-evacuated jack-ups, where potential life-safety and environmental pollution consequences have been mitigated, may be classified as exposure level L2. The requirements given in ISO 19900:2019, 7.3.3 shall be applied.

For extreme storm assessments, L2 jack-ups shall be assessed for the 50 year independent extremes or 100 year joint probability metocean data that can be reached at the site prior to evacuation being completed (e.g. 50 year sudden hurricane in tropical revolving storm areas) with allowance for forecast uncertainty, when appropriate. The assessment shall use the partial factors applicable to L1.

The slope of the sudden tropical storm hazard curve can be steeper than that of storms in other areas of the world and of full population tropical storms. The unoccupied post-evacuation case shall be considered in accordance with criteria to be agreed between the jack-up owner and the operator taking account of the slope of the sudden storm hazard curve and the time required to place the jack-up in the storm mode for the unoccupied condition to ensure adequate reliability of the jack-up in the sudden tropical storm ULS condition. Annex H.3 contains useful information on such conditions, developed for the US Gulf of Mexico, that could be used for other tropical storm areas. Any deviation from the storm mode given in the marine operations manual shall be clearly identified and agreed between the jack-up owner and the operator.

NOTE Operational and evacuation procedures are outside the scope of this document, however significant time can be required to prepare the jack-up for the unoccupied condition.

5.5.4 Exposure level L3

Jack-ups that meet the requirements for L2 and that are not normally occupied and may be classified as exposure level L3. The requirements given in ISO 19900:2019, 7.3.4 shall be applied.

For extreme storm assessments L3 jack-ups shall be assessed to criteria that shall be agreed between the jack-up owner and the operator.

5.5.5 Exposure level for earthquake

For earthquake, a jack-up shall be assessed as L1 unless it is normally unoccupied and meets the requirements given in ISO 19900:2019, 7.3.4, where item c) is changed to "visits are not planned to last more than 24 h on jack-ups in regions with seismic zones 1 to 4. When normally unoccupied, the earthquake assessment requirements shall be agreed between the jack-up owner and the operator.

5.6 Analytical tools

Guidance is given in ISO 19900:2019, Clauses 11 and A.11 on the use and validation of analytical tools and models. It should be noted that many software suites do not adequately address jack-up specific issues, such as time domain dynamics, foundations, large displacement effects and appropriate code checks.

6 Data to assemble for each site

6.1 Applicability

Clause 6 describes the data that are required to undertake an assessment. In this document, the field is the general area where the jack-up is to operate; the site is the specific position/orientation within the field. The site data are normally a subset of the field data. The data that should be included in the assessment report are listed in Annex G, which can be used as a check list.

6.2 Jack-up data

The jack-up data required to perform an assessment include the following:

— jack-up type;

— installed leg length;

— latest revision of the drawings, specifications and the operations manual;

— any proposed deviations from the operations manual limits for the intended operation;

— data pertaining to the strength, stiffness and operation of the leg-to-hull connection;

— proposed lightship and variable load and centres of gravity for each configuration, accounting for any changes that are not included in the latest revision of the operations manual;

— preloading capacity or pre-drive capability;

— limiting spudcan capacity, e.g. reactions and bearing pressure distribution(s) used in the design cases;

— design parameters including, where applicable, RPD limits

— details of any relevant modifications.

6.3 Site and operational data

The site data should include the site coordinates, sea floor topography and water depth referenced to a clearly specified datum, e.g. lowest astronomical tide (LAT) or chart datum (CD). Be aware that charts derived for use by comparatively shallow draft shipping are often not sufficiently accurate for siting jack-ups.

At platform sites, platform drawings, the required hull elevation or the required clearances with the platform, the jack-up heading and other interface data shall be obtained from the platform operator.

The assessor can use directional metocean data to optimize the jack-up heading. When directional metocean data are used in the assessment, the jack-up heading shall be specified. The overall reliability of the jack-up should not be compromised by the use of such criteria.

The data provided by the operator shall include the proposed mode of use (drilling, production, accommodation, etc.) and the number and size of any supported risers or conductors. The life-safety and consequence category of adjacent infrastructure while the jack-up is on site shall be provided.

6.4 Metocean data

It is of prime importance to obtain appropriate metocean data for the site with due recognition of the quality of the data. Site-specific data shall be obtained from or on behalf of the operator for the following:

a) water depth (LAT or CD);

b) tide and storm surge;

c) wave data:

  — significant wave height and spectral peak period (stating whether intrinsic or apparent, as discussed in A.7.3.3.5),

  — maximum wave height and associated period (stating whether intrinsic or apparent, as discussed in A.7.3.3.5),

  — abnormal wave crest elevation (see A.6.4.2.4).

d) current velocity and profile;

e) wind speed and profile.

Further reference to metocean data can be found in Table A.7.3-1.

Omnidirectional data can be sufficient but, in particular circumstances, directional data can also be required. Other data, such as the following, shall be evaluated, when applicable:

- marine growth distribution;
- icing;
- lowest average daily air temperatures, etc.

Directionality of wind, wave and current may be considered if accurate data are available. For deterministic analysis, wave kinematics factors may be applied to account for wave short-crestedness and jack-up leg spacing; see A.6.4.2.3.

General information on metocean data are given in ISO 19901-1. Details of the required metocean data for jack-up site-specific assessment are given in A.6.4.

Either the 50 year return period of individual extremes or the 100 year return period of joint probability metocean data shall be used for the site-specific assessment of occupied jack-ups. Partial action factors for the alternative return periods are given in 5.5.4, 8.8.1 and Annex B.

NOTE To provide consistent reliability levels, different action factors are used with actions determined for a 50 year return period of individual extremes and for a 100 year return period of joint probability metocean data.

As a minimum, a occupied-evacuated jack-up shall be assessed for the 50 year independent extremes or 100 year joint probability metocean data that can be reached while the jack-up is still occupied; see 5.5.4. For example in a TRS area, consideration may be given to the use of a 50 year return period “sudden hurricane”.

As a minimum, an unoccupied jack-up shall be assessed to an agreed exposure level; see Table 5.5-1.

If the jack-up deployment is to be of limited duration, applicable (seasonal) data may be used for the months under consideration, including suitable contingency.

6.5 Geophysical and geotechnical data

Site-specific geotechnical information applicable to the anticipated range of penetrations shall be obtained from or on behalf of the operator. The type and amount of geotechnical data required depend on the particular circumstances, such as the type of jack-up and previous experience at the site or nearby sites. Such information can include geophysical survey (sub-bottom profiler, side-scan sonar, bathymetry, magnetometer) data; boring/coring data; in situ and laboratory test data; and visual survey data.

The field shall be evaluated for the presence of geohazards. Such hazards and their potential mitigations are described in Table A.6.5-1.

For sites where previous operations have been performed by jack-ups of the same basic design, it can be sufficient to identify the location of the existing footprints, to assess the hazards associated therewith and refer to previous site data and preloading or penetration records; however, the accuracy of such information should be verified.

At sites where there is any uncertainty, borings/corings and/or in situ testing (e.g. piezocone penetrometer tests) data are recommended at the planned site. Alternatively, the site can be tied-in to such data at another site by means of shallow seismic data, although care must be taken to assess the uncertainty of any such extrapolation. If data are not available prior to the arrival of the jack-up, it can be possible to take boring(s)/coring(s), etc., from the jack-up before preloading and jacking to full hull elevation. Suitable precautions should be taken to ensure the safety of the jack-up during this initial period on site and during subsequent preloading. The newly acquired soils data shall be analysed and assessed to ensure foundation safety during and after preloading. The soil data should be used to update the site-specific assessment as necessary.

The site shall be evaluated for potential scour problems. These are most likely to occur at sites with high wave and/or current water particle velocity near a seabed that is composed of non-cohesive soils. See also 9.4.7.

Certain sites prone to mudslides can involve additional risks. Such risks should be assessed by carrying out specialist studies.

6.6 Earthquake data

Earthquake data shall be obtained through the use of ISO 19901-2.

6.7 Ice data

Ice data shall be obtained through the use of ISO 19906 and ISO 35106.

7 Actions

7.1 Applicability

This clause presents an overview of, and basic requirements for, the modelling of actions for site-specific assessment in accordance with this document.

Details regarding methods and formulations that can be applied to calculate actions are presented in A.7, which also includes presentation of hydrodynamic formulations and coefficients for detailed and equivalent modelling of hydrodynamic actions on legs.

In this clause and A.7, actions are presented as representative values. The representative actions shall be multiplied by the partial action factors as given in 8.8 prior to the determination of the assessment load cases.

7.2 General

The following outlines the actions that shall be considered in general terms:

a) metocean actions:

   1) actions on legs and other structures from wave and current;

   2) actions on hull and exposed areas (e.g. legs) from wind.

c) functional actions:

   1) fixed actions;

   2) actions from variable load.

d) indirect actions resulting from responses:

   1) displacement-dependent effects;

   2) accelerations from dynamic response.

e) earthquake actions;

f) ice actions;

g) other actions.

7.3 Metocean actions

7.3.1 General

Wind, wave and current actions are typically considered to act simultaneously and from the same direction. This colinearity should normally be assumed. The directionality of wind, wave and current may be considered when it can be demonstrated that such directionality is applicable at the site under consideration.

7.3.2 Hydrodynamic model

The hydrodynamic modelling of the jack-up leg can be carried out by utilizing “detailed” or “equivalent” techniques. The hydrodynamic models shall represent all structures and appurtenances subjected to wave and current action. The effect of different hydrodynamic properties in different directions shall be represented as appropriate for the analysis.

Hydrodynamic (drag and inertia) coefficients shall be selected that are appropriate for the flow regime of the actual jack-up leg structure and chosen wave theory. Applicable test results may be used to select the coefficients for non-circular members (and not the complete leg). The effects of raw water piping, ladders and other appurtenances shall be considered in the calculation of the hydrodynamic coefficients for the legs.

The effect of marine growth on the actions shall be considered. Because jack-ups are mobile, opportunities are available to clean the leg to reduce hydrodynamic actions.

7.3.3 Wave and current actions

Wave and current actions on the legs and appurtenances (e.g. raw water tower) shall be computed using the Morison equation and an appropriate hydrodynamic model. A wave theory appropriate to the wave height, period and water depth shall be used for the determination of particle kinematics. Wave kinematics for the calculation of actions caused by waves shall be derived from the intrinsic wave period or the intrinsic wave frequency.

NOTE When waves are superimposed on a (uniform) current, the intrinsic reference frame for the waves travels at the speed and in the direction of the underlying current. An observer travelling at the same speed and in the same direction as the current is stationary with respect to the intrinsic reference frame and, therefore, measures the intrinsic wave period (see A.7.3.3.5 and ISO 19901-1:2015, 8.4.4 and A.8.4.3). The wave has only an intrinsic wave length; there is no apparent wave length.

The derived actions are directly affected by the current profile chosen and the method used to modify the profile when the height of the water column varies in the presence of waves. Guidance is provided in A.6.4.3.

VIV is normally considered to be covered by class, but should be checked for jack-ups with large-diameter tubular legs when the current velocity exceeds that used in the design; see for example DNV-RP-C205 (DNV 2021d)[60]; Grundmeier, Campbell and Wesselink (1989)[85] and Blevins (1990)[26].

7.3.4 Wind actions

All structures and appurtenances subjected to wind action shall be considered. Wind actions shall be computed using wind velocity, wind profile and exposed areas. Appropriate wind velocities and wind profiles shall be used, guidance is given in A.6.4.6. These actions can be calculated using appropriate formulae and coefficients or can be derived from applicable wind tunnel tests. Generally, block areas are used for the hull, superstructures and appurtenances.

Wind actions on legs can be a dominant factor for jack-ups operating at less than their maximum design water depth.

The potential effects of wind-induced VIV should be considered, see for example DNV-RP-C205 (DNV 2021d)[60]; Grundmeier, Campbell and Wesselink (1989)[85] and Blevins (1990)[26].

7.4 Functional actions

For functional actions, it is usual to consider the jack-up with the maximum permitted variable load for structural checks and with the minimum anticipated variable load (often 50 %) for the overturning calculation. If the assessment of the jack-up shows that it is marginal in one of these configurations, consideration may be given to limiting the variable load to a lower or higher level (depending on the critical parameter), providing the jack-up can be successfully operated under such restrictions. The assessor shall document any restrictions on the variable load that apply to the operating limits at the site and communicate them to the jack-up owner. The intent is to ensure that these limits are included in the operating procedures for the site.

7.5 Displacement dependent effects

Indirect forces that are a consequence of the displacement of the structure and its foundation shall be considered in the analysis. The effects are due to the first-order sway, foundation settlement, and to the enhancement due to the increased flexibility of the legs in the presence of axial actions (Euler amplification); see A.8.8.6.

7.6 Dynamic effects

Indirect forces due to dynamic response of the jack-up shall be considered and are particularly important for sea states having significant energy near the natural periods of the jack-up or multiples thereof; see 10.5.2 and 10.5.3.

Dynamic effects shall be included in earthquake analyses (see 10.7) and can be important for ice action responses.

7.7 Earthquakes

Actions and action effects due to earthquakes shall be considered where appropriate; see 8.8.8 and 10.7.

7.8 Ice actions

Actions and action effects due to ice shall be considered where appropriate; see 10.8 and A.10.8.

7.9 Other actions

Additional leg moments due to leg inclination resulting from leg-to-hull clearances and hull inclination shall be considered as described in 8.3.6 and 10.5.4.

Other types of action, for example actions due to icing and snow or sudden drop due to reservoir subsidence can occur in certain geographical regions. These actions shall be computed and applied in combination with other appropriate concurrent actions.

8 Structural modelling

8.1 Applicability

This clause presents methods for the development of an analytical model of an independent leg jack-up structure. Included in a jack-up structure are the legs, hull, leg-to-hull connection, and spudcans. The modelling of the foundation is presented in Clause 9.

The modelling provisions cover the generation of stiffness, self-weight, mass and application of actions.

In this clause, and the corresponding A.8, values for actions, forces, reactions, masses, stiffnesses, moments and geometry are presented as representative values unless indicated otherwise. Actions shall be multiplied by the partial action factors as given in 8.8 prior to the determination of the assessment load cases.

8.2 Overall considerations

8.2.1 General

In general, structural modelling for the assessment of a jack-up shall achieve the following objectives for both static and dynamic responses:

— realistic global response (e.g. displacement, base shear, overturning moment) for the jack-up under the applicable environmental and functional actions;

— suitable representation of the leg, leg-to-hull connection and the leg-foundation interaction, including non-linear effects as necessary;

— adequate detail to enable realistic assessment of the leg structure, the structural/mechanical components of the jacking and/or fixation system and the foundation.

8.2.2 Modelling philosophy

The purpose of structural modelling is to estimate the forces and displacements in a structure when subjected to the calculated applied actions.

The distribution of global actions and estimates of internal forces and displacements can be obtained through the use of simplified, equivalent modelling techniques.

To determine displacements and forces in the leg, leg-to-hull connection, leg/spudcan connection and local hull displacements, a finite element (FE) model shall be developed.

An explicit model of the conductor is rarely warranted, however the loading from conductor(s) shall be included.

8.2.3 Levels of FE modelling

In general, a jack-up model shall include the leg, leg-to-hull connection and representative hull structure. FE models can contain combinations of detailed and simplified structural modelling. Four modelling techniques are summarized below, with further detail given in 8.3 through 8.6:

a) fully detailed model of all legs and leg-to-hull connections, with detailed or representative stiffness model of hull and spudcan;

b) equivalent leg (stick model) and equivalent hull; equivalent stiffness model of all legs and spudcans, equivalent leg-to-hull connection springs and representative beam-element hull grillage;

c) combined equivalent/detailed leg and hull; simplified lower legs and spudcans, detailed upper legs and leg-to-hull connections with detailed or representative stiffness model of the hull;

d) detailed single leg (or leg section) and leg-to-hull connection model. This model shall be used in conjunction with the reactions at the spudcan or the forces and moments in the vicinity of the lower guide obtained from model b).

8.3 Modelling the leg

8.3.1 General

The leg can be modelled as a “detailed leg”, an “equivalent leg” or a combination of the two.

8.3.2 Detailed leg

A “detailed leg” model consists of all structural members, such as chords, horizontal, diagonal and internal braces of the leg structure and the spudcan (if required). Each structural component of the leg is represented by one or more appropriate finite elements. In the development of a detailed leg model, the use of beam elements is generally accepted practice. However, other finite elements can be utilized, when necessary, to accurately represent individual structural members.

8.3.3 Equivalent leg (stick model)

An “equivalent leg” model consists of a series of collinear beam elements simulating the complete leg structure. In this model, a series of one or more beam elements represents the overall stiffness characteristics of the detailed leg.

8.3.4 Combination of detailed and equivalent leg

In this model, the areas of interest are modelled in detail and the remainder of the leg is modelled as an equivalent leg.

8.3.5 Stiffness adjustment

The leg stiffness used in the overall response analysis can account for a contribution from a portion of the rack tooth material. Unless detailed calculations indicate otherwise, the assumed effective area of the rack teeth should not exceed 10 % of their maximum cross-sectional area. When checking the strength of the chords, the chord properties should be determined discounting the rack teeth.

8.3.6 Leg inclination

The additional leg moment due to leg inclination resulting from leg-to-hull clearances and hull inclination shall be considered (see 10.5.4), but it is not necessary that it be explicitly modelled.

The designed-in leg inclination of slant-leg jack-ups shall be modelled explicitly.

8.4 Modelling the hull

8.4.1 General

The hull structure shall be modelled so that the actions can be correctly transferred to the legs and the hull flexibility is represented accurately.

8.4.2 Detailed hull model

The detailed hull model shall include primary load carrying structures, explicitly modelled with appropriate finite elements.

8.4.3 Equivalent hull model

If a detailed hull model is not used, an equivalent hull model shall be constructed using a grillage of beams.

8.5 Modelling the leg-to-hull connection

8.5.1 General

The leg-to-hull connection controls the distribution of leg bending moments and shears carried between the guides and the jacking/fixation system. In the elevated mode, the most heavily loaded portion of the leg is normally within the vicinity of the leg-to-hull connection. The model shall provide the means to identify any possible leg-to-hull contact at locations other than the guides.

8.5.2 Guide systems

The guide structures restraining the chord members shall be modelled, accounting for clearances and their direction of action. When chord-to-guide contact occurs in the span between chord-brace connections, significant local chord bending moments can occur. Therefore, various guide positions shall be investigated.

8.5.3 Elevating system

The elevating systems shall be modelled using either the stiffness derived from detailed analysis or from testing. Generally, the manufacturer specifies this information.

8.5.4 Fixation system

If the jack-up is equipped with a fixation system, e.g. rack chocks, it shall be modelled to resist both vertical and horizontal forces, using appropriate stiffnesses.

8.5.5 Shock pad — floating jacking systems

For floating jacking systems, the shock pad stiffness shall be modelled and the shock pad shall be modelled to resist vertical compressive forces only. Generally, the manufacturer specifies the stiffness information.

8.5.6 Jackcase and associated bracing

The jackcase or jackhouse structures and associated bracing shall be modelled based on their actual stiffness.

8.5.7 Equivalent leg-to-hull stiffness

The model shall represent the overall stiffness characteristics of the leg-to-hull connection.

8.6 Modelling the spudcan and foundation

8.6.1 Spudcan structure

The spudcan structure shall be modelled with sufficient detail to accurately transfer the seabed reaction into the leg structure.

Where there is insufficient data available regarding the structural strength of the spudcans, the suitability of the spudcans for the site shall be determined from applicable analyses.

8.6.2 Seabed reaction point

Selection of the reaction point shall be based on the penetrations (see 9.3.2) and shall consider any anticipated horizontal eccentricity.

8.6.3 Foundation modelling

For the analysis of an independent leg jack-up unit in the elevated storm mode, the foundations may be assumed to behave as pinned supports, which are unable to sustain moment. This is a conservative approach for the bending moment in the leg in way of the leg-to-hull connection.

In cases where the inclusion of rotational foundation fixity is justified and is included in the structural analysis, the non-linear soil-structure interaction effects shall be taken into account. The model shall include the interaction of rotational, lateral and vertical soil forces. Methods of establishing foundation fixity are given in Clause 9.

When fixity brings the structural natural period closer to the excitation frequency, the inclusion of foundation fixity can amplify the response and shall, therefore, be considered.

The spudcans, the leg-to-can connection and the lower parts of the leg are addressed at the design stage. In cases where the spudcan reactions could exceed the design values the reactions used to assess these areas shall be obtained from a foundation model that provides a high estimate of the spudcan moment.

For foundation modelling under earthquake excitation see 10.7 and A.10.7.

8.7 Mass modelling

The mass model shall reflect the mass distribution of the jack-up. The model shall include structural and non-structural mass, including entrapped fluids, marine growth, added mass, etc. The added mass shall be computed based on the displaced volume of the submerged components, including marine growth, acting in the direction of motion normal to the component. The mass of the variable load (e.g. consumables stored on/within the hull) shall be included in the mass model.

Some actions that are included in the variable load are not masses and shall not be included in the mass model (e.g. conductor tension and hook loads).

- The structural mass shall include:
  - legs;
  - hull structure;
  - spudcans.
- The non-structural mass shall include:
  - hull equipment and outfitting;
  - mass of the variable load;
  - sea water supply system;
  - leg appurtenances;
  - marine growth;
  - entrapped water in flooded members and spudcans.
- Added mass shall include contributions from:

— submerged legs and leg components, e.g. chords and braces;

— sea water caissons;

For earthquake assessments see 10.7 for additional guidance on the mass model.

8.8 Application of actions

8.8.1 Assessment actions

8.8.1.1 General

The assessment load case, F_d, shall be determined using the following generalized form in which the partial factors are applied before undertaking the structural response analysis to ensure that the non-linear behaviour is properly captured, as given in Formula (8.8-1):

$$F_d = \gamma_{f,G} G_F + \gamma_{f,V} G_V + \gamma_{f,E} (E_e + \gamma_{f,D} D_e) \tag{8.8-1}$$

where

G_F are actions due to the fixed load positioned such as to adequately represent their vertical and horizontal distribution; see 8.8.2;

G_v are actions due to maximum or minimum variable load, as appropriate, positioned at the most onerous centre of gravity location applicable to the configurations under consideration; see 8.8.2;

E_e are metocean actions due to the extreme storm event; see 8.8.4 ($E_e = 0$ for earthquake assessment);

D_e is an equivalent set of inertial actions representing dynamic extreme storm effects; see 8.8.5 ($D_e = 0$ for stochastic storm assessment in accordance with 10.5.3);

D_e is an equivalent set of inertial actions induced by the ELE or ALE ground motion for earthquake assessment; see 8.8.8;

γ are the partial action factors, as given in 8.8.1.2 to 8.8.1.4.

NOTE See Annex B, which contains all of the applicable factors for use in a site-specific analysis.

The actions and action effects that shall be included in the analysis are outlined in 8.8.2 to 8.8.8.

8.8.1.2 Two-stage deterministic storm analysis

The partial action factors for the deterministic storm analysis described in 10.5.2 and A.10.5.2.2.3 shall be as given below:

— $\gamma_{f,G} = 1,0$ and is applied to the actions due to fixed load;

— $\gamma_{f,V} = 1,0$ and is applied to the actions due to the variable load;

— $\gamma_{f,E} = 1,15$ when applied to the actions due to the 50 year return period independent extreme metocean data;

— $\gamma_{f,E} = 1,25$ when applied to the actions due to the 100 year return period joint probability metocean data;

— $\gamma_{f,D}$ = 1,0 and is applied to the inertial actions due to dynamic response.

8.8.1.3 Stochastic storm analysis

As discussed in A.10.5.3.2, in a stochastic storm analysis the metocean wind wave and current parameters are increased such that an action factor of 1,0 can be applied while achieving comparable global factored actions. Consequently, the stochastic storm analysis described in 10.5.3 is carried out using unfactored actions. The resulting partial action factors shall be as given below:

— $\gamma_{f,G}$ = 1,0 and is applied to the actions due to fixed load;

— $\gamma_{f,V}$ = 1,0 and is applied to the actions due to the variable load;

— $\gamma_{f,E}$ = 1,0 when applied to the metocean actions derived from the factored wind, wave and current metocean parameters, see 10.5.3, A.10.5.3;

— $\gamma_{f,D}$ = 1,0 and is applied to the inertial actions due to dynamic response.

8.8.1.4 Earthquake analysis

8.8.1.4.1 The partial action factors for ELE analysis described in 10.7 shall be as given below:

— $\gamma_{f,G}$ = 1,0 and is applied to the actions due to fixed load;

— $\gamma_{f,V}$ = 1,0 and is applied to the actions due to the variable load;

— $\gamma_{f,E}$ = 0,9 when applied to the ELE actions;

— $\gamma_{f,D}$ = 1,0 and is applied to the inertial actions induced by the ELE ground motion ($E_e = 0$).

8.8.1.4.2 The partial action factors for the ALE shall be as given below:

— $\gamma_{f,G}$ = 1,0 and is applied to the actions due to fixed load;

— $\gamma_{f,V}$ = 1,0 and is applied to the actions due to the variable load;

— $\gamma_{f,E}$ = 1,0 when applied to the ALE actions;

— $\gamma_{f,D}$ = 1,0 and is applied to the inertial actions induced by the ALE ground motion ($E_e = 0$).

NOTE The apparent inconsistency between the earthquake partial action factors is due to the differences in the analysis methods used for the ELE and ALE assessments. The 0,9 partial action factor in conjunction with the normal resistance factors is taken from ISO 19902. The 0,9 partial factor was determined in the API calibration of LRFD against WSD. The ALE action factor of 1,0 is used in conjunction with a system survival assessment.

8.8.2 Functional actions due to fixed load and variable load

8.8.2.1 The actions due to fixed load (i.e. hull, legs, outfit, stationary and movable equipment) include:

— weight in air including appropriate solid ballast;

— weight of permanent enclosed liquid;

— buoyancy.

8.8.2.2 The actions due to variable load, which comprises supplies or equipment that are expendable, readily removable, or consumable during operations, include:

— weight of liquid and solid stores;

— applied drilling and conductor loads;

— weight of readily removable equipment.

The actions due to fixed load and variable load shall be modelled to represent the correct vertical and horizontal weight and mass distribution.

8.8.3 Hull sagging

Hull sagging resulting from distributed actions and hull flexibility can impose bending moments on the legs. It shall be verified that the amount of hull sag-induced moment transferred to the legs in the analytical model is appropriate given the operating procedures of the jack-up and site-specific conditions.

8.8.4 Metocean actions

Wind actions on the legs and hull shall be modelled to represent their vertical and horizontal distribution.

Wave/current actions on the leg and spudcan structures above the sea floor shall be modelled to represent their vertical and horizontal distribution.

8.8.5 Inertial actions

The application of inertial actions depends on the dynamic approach adopted; see Clause 10. For the SDOF approach, the inertial actions are applied as horizontal force(s) acting through the hull centre of gravity. For deterministic storm analysis, with dynamics from a stochastic analysis, the forces are distributed to better approximate the dynamic overturning moment. Inertial actions should not normally be applied on the legs below the hull.

8.8.6 Large displacement effects

P-Δ effects occur because the jack-up is a relatively flexible structure and is subject to lateral displacement of the hull (sideways) under assessment actions (see 7.5).

P-Δ effects shall be included in the structural analysis.

8.8.7 Conductor actions

An explicit model of the conductor is rarely warranted. However, the top tension and actions on the jack-up due to the factored hydrodynamic actions on the conductor(s) shall be included in the analysis, if applicable.

8.8.8 Earthquake actions

See 10.7 and A.10.7 for earthquake actions.

8.8.9 Ice actions

See 10.8 and A.10.8 for ice actions.

9 Foundations

9.1 Applicability

This clause addresses the geotechnical considerations, soil-structure interaction, capacity, stiffness and hazards associated with the foundations that support independent leg jack-ups. Additional supporting information can be found in ISO 19901-4, however the provisions of this document shall take precedence in case of conflict.

In this clause, and the corresponding A.9, values for actions, forces, loads, preload, reactions, resistances, capacity, moments, weights and geometry are presented as representative values unless indicated otherwise. The representative actions shall be multiplied by the partial action factors as given in 8.8 prior to the determination of the assessment load cases.

NOTE The foundations of mat-supported jack-ups are not specifically covered in this document.

9.2 General

Adequate geotechnical and geophysical information as outlined in 6.5 shall be gathered and used to assess the spudcan penetration and foundation stability of the jack-up at the site. See further guidance in A.6.5. Applicable information from previous operations, other surveys or activities in the area should be used in the assessment of the site.

There are two objectives of gathering geotechnical and geophysical information. The first is to ensure that the foundation is adequate to carry static, cyclic, and transient forces without excessive settlement or movement. The second objective is to provide adequate information for foundation models of increasing sophistication for use in structural response analyses.

The assessment shall consider:

— the possible range of predicted leg penetrations;

— the possibility of rapid leg penetration and/or punch-through;

— likely scale of spudcan movements, e.g. due to consolidation, capacity exceedance;

— the effects of cyclic loading;

— the consequences of specific site conditions, such as are listed in 9.4.

9.3 Geotechnical analysis of independent leg foundations

9.3.1 Foundation modelling and assessment

The purpose of preloading is to develop adequate foundation capacity to resist the forces on the foundation due to assessment events. During preloading, the jack-up should normally be capable of generating spudcan reactions in excess of the maximum vertical reactions due to the factored actions determined in the assessment. Where the preload is insufficient to meet the Level 2 assessment criteria, such preload can be acceptable, e.g. if justified by the Level 3 displacement check in 9.3.6.

In some circumstances, the foundation capacities and stiffnesses from 9.3 are not sufficient for the unit to satisfy the acceptance criteria (Clause 13) based on the preload to be applied. In such cases the assessment can be based on foundation capacities and stiffnesses calculated using soil strength parameters and partial material factor γ_m instead of the applied preload. In such cases the requirements of 9.3 should be supplemented by the guidance and criteria for applicability in E.4.

The forces imposed on the foundation due to environmental actions are time-varying and random in nature. The response to the horizontal, vertical and rotational forces on the spudcan and the embedded portion of the leg is non-linear and hysteretic. The non-linearity of the foundation response can have a major effect on the response of the structure.

Two types of structural response analyses use a range of foundation models and are carried out as described in 10.4.4. These foundation models can include major simplifications and the limitations of the models should be understood by the assessor.

The foundation behaviour under the action of combined forces is appropriately described by a theoretical yield surface in the vertical reaction, horizontal reaction and moment reaction (VHM) space. Foundation safety assessment is achieved by comparing the imposed forces with the yield surface.

However, for structural response analysis, the foundation can be modelled as pinned or with a degree of foundation fixity. Foundation fixity is the rotational restraint offered by the soil supporting the spudcan and shall only be used in a model that also includes finite vertical and horizontal foundation stiffnesses. The degree of fixity is dependent on the soil type, the maximum vertical spudcan reaction during installation, the foundation stress history, the structural stiffness of the jack-up, the geometry of the spudcan, the spudcan translational and rotational displacements, and the simultaneous vertical and horizontal actions.

The structural response analysis shall be carried out using one of the following foundation models, which have increasing levels of complexity:

— pinned model: simple pinned foundation for all legs;

— secant model: linear vertical, linear horizontal and secant rotational stiffness where the iterative reduction of rotational stiffness ensures conformity with the yield interaction surface;

— yield interaction model: non-linear vertical, horizontal and rotational stiffness model where the non-linear behaviour ensures conformity with the yield interaction surface;

— continuum model: non-linear continuum foundation model coupled to the structure; this model shall also account for the load-penetration behaviour beyond the penetration achieved by preloading.

The assessment procedures for each of these models are described in 9.3.6.

9.3.2 Leg penetration during preloading

The methods for calculating ultimate vertical bearing capacity of a foundation in various types of soil are discussed in A.9.3.2. The gross bearing capacity formulae adopted are based on the assumption that penetration in sand is a drained process, and penetration in clay is an undrained process. Cases that deviate from this assumption shall be assessed using appropriate methods. Uncertainties regarding the geotechnical data should be properly reflected in the interpretation and reporting of the analyses. For the special case of carbonate material, see 9.4.10 and A.9.4.10.

The predicted spudcan penetration is obtained from the bearing capacity versus spudcan penetration curve at the specified preload. Soil backfill directly above the spudcan, composed of backflow and infill, shall be included when computing the penetration.

The use of predicted leg penetrations during jack-up deployment provides essential information on the compatibility between theoretical assessment and operational reality. Where there is significant deviation, the validity of the site-assessment should be re-evaluated.

9.3.3 Yield interaction

The yield interaction surface is used to describe the limiting combinations of vertical, horizontal and moment loading that the soil at a given penetration depth can sustain without becoming fully plastic. When the yield surface is transgressed, plastic deformation occurs and the spudcan reactions are redistributed.

During preloading, a significant volume of soil below the spudcan is made to plastically deform as the spudcan penetrates, thus generally expanding its yield surface and increasing its capacity. During removal of the preload, the soil unloads elastically and the foundation response is stiffer than during preload penetration. Provided the jack-up's preload capacity is appropriate for a site's environmental conditions, the soil behaves in an essentially elastic manner for most combinations of vertical, horizontal and moment loading that the spudcan experiences while on site. Inelastic response occurs when the combination of vertical, horizontal and moment loading approaches the yield surface; this is likely only for a few, if any, loading cycles during an extreme storm. Degradation can take the form of a softened foundation and/or additional displacement (vertical, horizontal, and/or rotational).

The yield surface can be described by the formulae given in A.9.3.3 for a range of soil types and embedments. The weight of all soil backflow and infill on top of the spudcan shall be included in the spudcan vertical reaction to be assessed against the yield surface.

For the case of layered soils, additional analysis should be performed to determine the appropriate yield surface.

9.3.4 Foundation stiffnesses

Foundation analysis under time-varying loading requires knowledge of the load-deflection behaviour of the soil. This is usually described by spring stiffnesses in the vertical, horizontal and rotational modes. Initial stiffnesses, as described in A.9.3.4.1, can be estimated from the solutions for a rigid circular plate on an elastic half-space using the small strain shear moduli for clay (see A.9.3.4.3) or sand (see A.9.3.4.4) and Poisson's ratio; alternatively, a continuum model can be used. The soil shear modulus is dependent on strain level; therefore, suitable adjustments should be made for cyclic and dynamic loading.

The reduction in stiffness as the spudcan reactions approach or exceed the yield surface shall be included in the analysis. There are different approaches to determining the softening of the stiffnesses. Where the reduction of stiffness is not included in the soil model, the provisions of A.9.3.4.2.2 should be used to determine the reduced rotational secant stiffness; the vertical and horizontal stiffness remain unchanged. The stiffness reduction is implicit in fully coupled yield interaction models and in non-linear continuum foundation models, as discussed in A.9.3.4.2.3 and A.9.3.4.2.4, respectively.

When the foundation is comprised of layered soils, additional analysis should be used to determine the effective stiffnesses.

The effects of soil-leg interaction for deep penetrations can be included. Guidance is given in A.9.3.4.6.

9.3.5 Vertical-horizontal foundation capacity envelopes

When the foundation is represented with the pinned or secant models, the spudcan reactions shall be assessed using the vertical-horizontal capacity envelopes. For the secant model, this assessment shall be performed after achieving conformity with the yield interaction surface. Spudcan reactions resulting from responses based on a model with pinned foundations for all legs may be assessed using the simplified preload and windward leg checks, provided that the individual spudcan reactions satisfy the associated applicability requirements.

The envelopes should be developed using the applicable subclause of A.9.3.5. The weight of all soil backfill that occurs during preloading shall be included in the spudcan vertical reaction when evaluating the capacity envelopes. Backfill after preloading shall be considered when its effect is to increase the foundation utilizations.

9.3.6 Acceptance checks

The overall jack-up foundation stability shall be assessed for the forces F_H and F_V, and the moment F_M, acting on each spudcan due to the assessment loading case F_d, using Levels 1, 2 or 3, as listed below (in order of increasing complexity and reducing conservatism); see Figure A.9.3-17. If a lower level check fails to meet the foundation acceptance criteria given in A.9.3.6, a higher level check can be performed. The partial factors for the checks required by this subclause are given in Annex B.

a) Level 1: Preload and windward leg check with reactions from a response analysis based on a pinned spudcan model for all legs; Steps 1a and 1b shall both be completed for a Level 1 check:

- Step 1a: Foundation capacity check of the leeward leg based on the preloading capability (A.9.3.6.2), and

- Step 1b: Check of the windward leg (A.9.3.6.3).

b) Level 2: Foundation capacity checks. One of the following three steps shall be completed for a Level 2 check:

- Step 2a: Foundation capacity check and sliding resistance check (A.9.3.6.4), based on the vertical and horizontal reactions, assuming a pinned spudcan; or

- Step 2b: Foundation capacity check and sliding resistance check (A.9.3.6.5), based on the vertical, horizontal and moment reactions from a spudcan model that includes rotational, vertical and horizontal foundation stiffness with rotational stiffness reduction; or

- Step 2c: Foundation capacity check (A.9.3.6.5), based on the vertical, horizontal and moment reactions from a spudcan model that includes rotational, vertical and horizontal foundation stiffness with reduction of vertical, horizontal and rotational stiffnesses. A Level 3 displacement check shall be performed.

c) Level 3: Displacement check (A.9.3.6.6). One of the following two steps shall be completed for a Level 3 check:

- Step 3a: Simple check using the leg-penetration curve based on the results of a Level 2 check when the foundation capacity check fails and/or a check of the effects of windward leg sliding when the Level 2 sliding check fails; or

- Step 3b: Numerical analysis of the complete jack-up and non-linear foundation coupled in vertical, horizontal and rotational degrees of freedom, e.g. finite element approach.

The maximum vertical reaction is expected to occur on the leeward leg. Likewise, the minimum vertical reaction is expected on the windward leg.

In Step 1a, the preload check of the leeward leg is based on the assumption that the net ultimate vertical bearing capacity is equal to the maximum spudcan reaction during preloading. Care shall be taken to account for the submerged weight of any backfill, $W_{BF,A}$ that occurs after the maximum preload has been applied. Typically backflow and infill after preloading, $W_{BF,A}$ is uncertain; for this reason, it should conservatively be included on the leeward leg but not on the windward leg. The check of the windward leg shall be performed to ensure that the sliding resistance is adequate under minimum vertical reaction conditions.

In Step 2a, the combined vertical and horizontal forces on the spudcan shall be checked against the factored vertical-horizontal foundation capacity and the factored sliding capacity of all legs. The vertical bearing capacity of the foundation is a function of the horizontal forces and moments. The sliding capacity of the foundation is a function of the vertical forces and moments. However, the moments are ignored in Step 2a analyses as the spudcans are considered to be pinned.

For Step 2b, the combined vertical and horizontal forces on the spudcan shall be checked against the factored vertical-horizontal foundation capacity envelope and the factored sliding capacity of all legs. The reactions are determined for a spudcan with "fixity" conditions whereby the interaction of moment with vertical and horizontal reactions is implicitly included through the use of the yield function.

For Step 2c, the foundation capacity and sliding checks are performed implicitly through the use of an unfactored yield function as described in A.9.3.3.

When a Step 2a or 2b assessment results in calculated factored combined vertical and horizontal forces on the spudcan that lie outside the factored bearing capacity envelope, a Level 3 assessment shall be used to evaluate the associated displacements. For all Step 2c analyses, a Step 3a assessment shall be performed. The procedure shall account for the redistribution of forces resulting from the overload and displacement of the spudcan(s). The acceptability of structural utilizations, overturning utilizations, foundation utilizations and displacements shall be re-evaluated in accordance with the acceptance criteria in Clause 13. The resulting displacement of the jack-up shall neither lead to the possibility of contact with any adjacent structure nor exceed practical limitations for continued operations.

Step 3a shall be accomplished by using the load-penetration curve to estimate the additional settlement for leeward legs. Sliding of windward legs shall be investigated. Additional settlement and sliding cause the magnitude and distribution of the foundation reactions to change. The effects on the structure shall be evaluated, including displacement dependent effects. If the effects are significant the procedure shall be iterated.

Step 3b shall be performed using a structural model including non-linear response of soil and structure (large displacement effects).

9.4 Other considerations

9.4.1 Skirted spudcans

Special consideration shall be given to the analysis of skirted spudcans including, but not limited to:

— skirt penetration;

— filling of any voids within skirt should partial penetration occur;

— bearing capacity (which can exceed preload, see E.4);

— settlement, including consolidation of trapped soils;

— moment capacity;

— sliding resistance;

— foundation stiffness;

— drainage paths;

— resistance to extraction;

— soil trapped within the skirt after extraction.

9.4.2 Hard sloping strata

Problems associated with positioning of spudcans on a hard sloping stratum at or below the sea floor shall be carefully considered. In this respect, a hard stratum is a soil layer where only partial spudcan penetration is expected and can be either a surface or a buried feature. Where a spudcan partially penetrates into a hard sloping stratum, there is potential to generate eccentricity in the spudcan reaction, which should be taken into account. There is also increased potential for slippage on sloping or undulating strata.

9.4.3 Footprint considerations

The depressions in the sea floor, or in harder layers within the seabed, that remain when a jack-up is removed from a site are referred to as footprints. The form of the depression depends on several factors such as the spudcan shape, the soil conditions, the spudcan penetration achieved and the method of extraction. The shape and the time period over which the depression exists can also be affected by the local sedimentary regime.

The positioning of spudcans very close to, or partially overlapping, footprints shall be carefully considered. This is because of the difference in resistance between the original soil and the disturbed soil in the footprint area and/or the slope at the footprint perimeter. The resulting leg displacements and/or eccentric spudcan loading can cause damage to the jack-up. The situation can be complicated by the proximity of a fixed structure or wellhead. The interaction between a spudcan and a footprint is expected to be minimal when the edge-to-edge distance exceeds one spudcan diameter, see Stewart and Finnie (2001)[173], Cassidy et al. (2009)[48], Gaudin et al. (2007)[78] and Gan et al. (2008)[77].

9.4.4 Leaning instability

Leaning instability of jack-ups can occur during operations in soft clays where the rate of increase in bearing capacity with penetration is small, leading to uncontrollable leg penetration. The potential for and consequences of such instability shall be considered.

9.4.5 Leg extraction difficulties

Prior to emplacement of the jack-up, consideration shall be given to potential leg extraction difficulties; see A.9.4.5.

9.4.6 Cyclic mobility, liquefaction and liquefaction-induced lateral flow

Cyclic loading can cause a progressive build-up of pore pressures within the foundation soils and consequent soil strength degradation (cyclic mobility or liquefaction). The effects can be either over a large area or local to the soils under the spudcan.

Earthquakes cause cyclic loading in the soil and can result in failure of the soil mass locally or over a large area. At a site with, or adjacent to, a sloping seabed, the potential for earthquake induced large-scale liquefaction-induced lateral flow that could affect the jack-up should be assessed; if present, the site should be rejected.

Local foundation cyclic loading can be caused by the jack-up response to earthquakes, severe storms, rotating machinery, etc. Depending on the magnitude of pore pressures developed, cyclic loading can result in large vertical and lateral displacements of the spudcans, which can be differential in some cases. The assessment shall consider the effects of cyclic loading on the stability and displacements of foundations. Guidance is provided in A.9.4.6.

9.4.7 Scour

When a spudcan is installed on the sea floor, its presence can cause increased local flow velocities (due to wave and current) that can result in the sea floor soils being eroded. The phenomenon of scour is observed around spudcans that are embedded in granular materials at sites with high sea floor flow velocities. If scour is recognized to potentially cause problems, then preventive measures shall be implemented. See A.9.4.7 for further guidance.

9.4.8 Spudcan interaction with adjacent infrastructure

For jack-ups located in close proximity to pile-founded structures, soil displacements caused by the spudcan penetration can induce actions on the nearby piles. The magnitude of the soil displacement depends on the spudcan proximity (distance of the spudcan edge to the pile's outside surface), the spudcan diameter, penetration, and soil stratigraphy. If the proximity of the spudcan to the pile is greater than one spudcan diameter, then no significant lateral actions on the pile are expected in a homogeneous single-layer soil system. However, this is not necessarily true for a layered soil system. When the proximity is less than one spudcan diameter or layered soil conditions are encountered, then the assessor should report the possibility of induced actions on the pile(s).

Guidance regarding the analytical procedures available for assessing these spudcan induced actions on piles, pipelines and other adjacent infrastructure is given in A.9.4.8.

9.4.9 Geohazards

Natural, shallow geological features and conditions such as faults, scarps, fluid expulsion features and gas-charged or over-pressurized sediments can pose additional threats to jack-ups that are independent of the forces on the foundation. These geological hazards, collectively called geohazards, can result in unforeseen events such as submarine slides and uncontrolled fluid releases that can adversely affect jack-up performance and/or stability. These events can be triggered by natural phenomena such as earthquakes or by human activities such as drilling.

Shallow geohazard risk assessments are performed routinely in the offshore industry to safeguard well and geotechnical drilling operations from subsurface hazards such as shallow gas. However, it is important that a pre-installation shallow hazard assessment for a jack-up consider the overall geological setting and all the geohazards that can threaten the jack-up or its operations while on site. This work should be conducted and assured by competent geohazard specialists. Further information is given in A.9.4.9.

9.4.10 Carbonate material

Carbonate materials can exhibit unexpected behaviour and should be addressed with care (see A.9.3.2.5 and ISO 19901-4).

10 Structural response

10.1 Applicability

The response of a jack-up is determined by applying actions in accordance with the assessment load case F_d (see 8.8) to the structural model to determine displacements, internal forces in components and reactions at the foundations. Responses shall be compared with resistances to determine the utilization of the jack-up structure and its foundation; acceptance criteria are given in Clause 13.

This clause presents methods for calculating the response of a jack-up including static and dynamic effects. This clause also presents a discussion of the important parameters affecting the dynamic response, including mass, stiffness and damping. Actions are presented in Clause 7. Stiffness and mass modelling and the application of actions are addressed in Clause 8. Foundation modelling is addressed in Clause 9.

10.2 General considerations

Action effects required for the assessment of jack-ups in the ULS typically include:

— component forces that shall be checked to determine the adequacy of individual structural components;

— foundation reactions that shall be checked to determine foundation performance and global stability;

— displacements to check for interaction with adjacent structures.

Action effects required for the assessment of jack-ups in the FLS, when applicable for long-term operations, typically include local cyclic stresses which shall be checked to assess fatigue damage (see Clause 11).

10.3 Types of analyses and associated methods

A jack-up shall be assessed for the in-place elevated storm mode. Depending on the geographic location of the site, assessments for earthquake, ice and abnormal environmental events can be required. In unusual circumstances, assessments for fatigue resistance and accidental situations can be required.

Different methods of analysis can be used for the various limit states to be considered. The methods of analysis for the in-place elevated storm mode include:

— deterministic two-stage analysis, in which the responses of the jack-up are determined by analysing a single combination of actions for each assessment situation;

— stochastic one-stage analysis in which extreme values of the responses of the jack-up are determined statistically by analysing multiple combinations of (environmental) actions for each assessment situation. Because of the inherent non-linearity of jack-ups, stochastic analyses are performed in the time domain;

— ultimate strength analysis in which the collapse strength of the jack-up structure and its foundation are determined.

Table 10.3-1 summarizes the analysis requirements for different assessment situations. The analyses shall consider the parameters discussed in 10.4.

Table 10.3-1 — Analysis requirements for different assessment situations

In-place elevated mode	Deterministic analysis		Stochastic analysis		Ultimate strength analysis
	Linear	Non-linear	Dynamic linear	Dynamic non-linear	
Ultimate and serviceability limit states (ULS and SLS)	See 10.5, A.10.5.2 and A.10.5.3				Generally outside the scope of this document. See 10.10
Fatigue limit state (FLS)	See 10.6	not applicable	See 10.6	not applicable	not applicable
Accidental/Abnormal limit state (ALS)	Appropriate, but can be unduly conservative	Appropriate, but outside the scope of this document	Appropriate, but can be conservative	Appropriate, but outside the scope of this document	Generally outside the scope of this document. However see 10.8 for ice
Earthquake (ULS or ALS)	See 10.7 and A.10.7		Appropriate, but outside the scope of this document		Generally outside the scope of this document. See A.10.7.4

10.4 Common parameters

10.4.1 General

A description of important parameters that are applicable to all analysis methods is given in 10.4.

10.4.2 Natural periods and related considerations

10.4.2.1 General

The estimation of natural periods is critical for the determination of the structural responses because jack-ups can exhibit significant dynamic effects. As a result, the dynamic responses can differ markedly from the static responses. The assessment of responses shall consider the possible variation of the natural periods and its implication on the accuracy of the analyses.

Determining the natural periods depends upon accurate estimates for

- the water depth and hull elevation,
- leg penetration and nature of the foundation, and
- the magnitude and location of masses associated with actions due to fixed load and variable load.

10.4.2.2 Stiffness

The overall stiffness of the jack-up shall be determined including the hull, legs, leg-to-hull connection, foundation and the P-Δ geometric effects as defined by the modelling practices in Clause 8. A range of stiffness values should be considered if stiffness information is not well defined.

10.4.2.3 Mass

The mass model shall include contributions from structural, non-structural and added masses (see 8.7).

For all analysis types, the most likely mass distribution should be considered, e.g. the position of the cantilever, the distribution of the variable load, and the level of marine growth. A range of values or distributions should be considered if mass information is not well defined or when the tolerances on the known position are significant.

10.4.2.4 Variability in natural period

The variability in natural period shall be considered. There are several factors that can cause variability in natural periods including stiffness non-linearities in the structure and foundation. The natural periods of the jack-up are a function of the static and time-varying response due to non-linearities in the structural and foundation behaviour. Structural non-linearities can result from stiffness changes (gap impact, yielding, etc.). Foundation non-linearities can result from changes in stiffness as a function of the force level with respect to the yield surface and force reversal (hysteresis). For example, the variability in natural period should be taken into account when selecting the levels of fixity to use in the analysis as it can affect the influence of wave reinforcement and/or cancellation effects.

NOTE The calculated natural periods can vary considerably between linear elastic and non-linear analyses.

10.4.2.5 Cancellation and reinforcement

Cancellation is the situation where, due to the spacing between the jack-up legs with respect to the wave length, the wave action on the jack-up is close to zero over the complete wave cycle. The primary parameters for reinforcement and cancellation effects are the wave length and the leg spacing. First cancellation occurs when the crest and trough of the same wave cycle are at two legs (leg spacing one half of the wave length). First reinforcement occurs when the crests of successive wave cycles are at the legs. Subsequent order period cancellations and reinforcements occur at progressively shorter periods.

The wave period used in the deterministic extreme storm analysis shall be chosen with the range to minimize the effects of cancellation, see e.g. A.6.4.2.3.

In a random wave dynamic analysis, wave action cancellation can significantly reduce the dynamic amplification. This effect should be minimized by adjusting the natural period of the jack-up to be away from the cancellation periods.

10.4.3 Damping

Contributions to the system damping include foundation damping, hydrodynamic damping and structural damping. Non-linear behaviour of the foundation and the jacking system also contributes to system damping. The degree to which each of these contributions affects the system damping depends on the type of analysis and the level of system response.

10.4.4 Foundations

The analysis of the structure and the assessment of the foundation can be performed essentially in two different ways.

- Option 1: Deterministic two-stage approach. The first stage is to calculate the dynamic amplification factor and inertial loadset, often using linearized analyses. The foundation and structural assessment is then performed using a quasi-static iterative or elasto-plastic analysis technique, for which the dynamic actions are approximated by the pre-determined inertial loadset.
- Option 2: Stochastic one-stage approach, where dynamic structural analysis and assessment is performed using one model. Here, a fully detailed non-linear time domain stochastic analysis is performed taking into account the elasto-plastic behaviour of the foundation.

10.4.5 Storm excitation

Wind, current and waves all contribute to the storm excitation. The primary source of dynamic excitation is from the fluctuating nature of waves.

As waves and currents interact, these two metocean factors should be considered in combination when generating time-varying hydrodynamic actions in accordance with Clauses 7 and A.7.

Various mean wave directions shall be considered. The effect of wave spreading around the mean direction may be taken into account, provided reliable information is available.

When using joint probability metocean data, relevant combinations of wind, waves and current shall be considered to determine the most onerous combination (see A.7.3.1.1).

Sea states with a peak period close to the natural period of the jack-up can give larger dynamic amplification resulting in larger responses in lower sea states than the extreme storm event. Therefore, waves with peak periods close to the natural period of the jack-up should be considered (see A.6.4.2.9).

10.5 Storm analysis

10.5.1 General

A jack-up responds dynamically to time-varying wave actions (see 10.4.5 and A.10.4.5). This behaviour shall be modelled appropriately in the analysis by including the static and dynamic contributions. These effects can be determined by a two-stage deterministic or by a one-stage stochastic analysis procedure.

Static actions due to fixed loads, variable loads and wind actions shall be combined with the time-varying wave and current actions.

A two-stage deterministic storm analysis involves developing static metocean actions and an inertial loadset. The inertial loadset can be developed from either a classical SDOF analogy or from a random dynamic analysis, in both cases through the development of a DAF (see 10.5.2). The inertial loadset shall be applied to be in phase with, and to increase the response to, the metocean actions as one of the loadcases. When the natural period divided by the apparent wave period is greater than 0,9, caution shall be exercised and additional loadcases for different inertial phases should be considered.

A more detailed time domain stochastic storm analysis procedure, in which inertial actions are directly included, can also be used. This analysis predicts the combined static and dynamic response of the jack-up to random wave actions from which the most probable maximum extreme (MPME) responses are calculated; see 10.5.3.

Action effects due to leg inclination shall be combined with action effects due to the extreme storm event to maximize leg and holding system strength utilizations.

Table 10.5-1 summarizes the two approaches to incorporating foundation response (10.4.4) and dynamics in the analysis.

10.5.2 Two-stage deterministic storm analysis

The most common method of analysis adopted for the determination of the extreme response is the deterministic, quasi-static wave analysis. This method does not reflect the random nature of wave excitation and assumes that the extreme responses are uniquely linked to the occurrence of a single and periodic extreme wave.

Deterministic responses are normally calculated by time stepping the single and periodic extreme wave through the structure. The extreme responses are determined from the following:

— the actions due to fixed loads, variable loads and wind actions;

— the time-dependent, but quasi-static wave/current actions;

— an inertial loadset representing dynamic effects.

The actions of the first and second list items above shall be determined in accordance with Clause 7.

Table 10.5-1 — Methods of extreme storm analysis

Parameter	Two-stage deterministic storm analysis			One-stage stochastic storm analysis
	Stage 1 Determine DAF		Stage 2 Single deterministic storm analysis	Multiple random time domain simulations
	$K_{DAF,SDOF}$	$K_{DAF,RANDOM}$		
Wave/current actions	not applicable	Random (superposition of linear components)	High order regular wave	Random (linear or higher order)
Dynamics	Formula (A.10.5-1) (see A.10.5.2.2.2)	Time domain simulations (see A.10.5.2.2.3)	Inertial loadset determined by means of $K_{DAF,SDOF}$ or $K_{DAF,RANDOM}$ (see A.10.5.2)	Time domain simulations (see A.10.5.3)
Wind actions	not applicable	Ignore	Quasi-static	Quasi-static
Foundation	Linearized	Linearized	Non-linear	Non-linear
Structure	Stiffness from non-linear structure	Non-linear or calibrated to non-linear	Non-linear	Non-linear
Output	$K_{DAF,SDOF}$	$K_{DAF,RANDOM}$	(Global) responses	(Global) responses

The inertial actions induced by time-varying wave and current actions are approximately represented by an inertial loadset. The magnitude of the inertial loadset is determined from a DAF and the quasi-static wave/current actions. Methods of calculating the DAF include:

- a classical single degree-of-freedom analogy;
- determining the ratio of dynamic and quasi-static responses from random dynamic analyses.

A.10.5.2.2.3 gives load cases that should be considered when $K_{DAF,\ RANDOM}$ is used to determine the inertial loadset in a two-stage analysis. The first load case that includes in-phase inertial load shall always be considered, e.g. as Formula (A.10.5-4). When $(T_n/T_p) > 0{,}9$, additional load cases considering out-of-phase inertial loads should be considered, e.g. the three shown in A.10.5.2.2.3, Formulae (A.10.5-5) to (A.10.5-7).

When determining DAFs, P-Δ effects shall be included in both the quasi-static and the dynamic analyses and the contribution of the P-Δ effect to the overturning moment shall be included in the overturning moment.

10.5.3 Stochastic storm analysis

In the stochastic method, one or more random dynamic analyses are performed for a given sea state or for a range of sea states. As the stochastic wave and current excitation varies with multiple realizations of a sea state, the extreme responses in each realization also vary. The most probable maximum extreme response can be determined through statistical analysis of one or more simulations.

In each simulation, the actions due to fixed loads, variable load and wind actions are combined with the time-varying wave/current actions. The actions shall be determined in accordance with Clause 7. The influence of dynamic effects is inherently included in the results of the dynamic stochastic analyses.

When undertaking a fully integrated dynamic stochastic analysis that directly results in a time history of structural and foundation utilizations, it is necessary to determine the MPME of each utilization.

The action factors on metocean actions for this analysis method shall be set to 1,0 in accordance with 8.8.1.3. To obtain a consistent level of environmental actions the metocean parameters (i.e. wind velocity, wave height and current velocity) shall be factored; see A.10.5.3.

NOTE The inclusion of action factors not equal to unity is complex and open to physical inconsistencies and misapplication. The more logical approach of applying partial factors to the metocean parameters has been adopted for fully integrated dynamic stochastic analyses. However, the partial factors on metocean parameters for stochastic analysis used for determining the DAF are set to unity.

10.5.4 Initial leg inclination

The initial leg inclination resulting from guide clearances and from the permitted hull inclination results in additional leg moment. If the initial leg inclination is explicitly modelled, the additional moments are inherently included in the results.

If the initial leg inclination is not explicitly modelled, the member forces and holding system forces from the analysis in accordance with 10.5.2 or 10.5.3 shall be increased to account for the effect of the additional leg moment prior to undertaking the structural strength checks; see A.10.5.4.

In all cases, the direction of the moment shall be such as to maximize the utilization checks in the vicinity of the hull; this can be achieved simply by considering the base of the legs to be offset in the up-wind direction.

10.5.5 Limit state checks

Limit state checks shall be performed for:

— strength of leg members, particularly in the vicinity of the upper and lower guides and adjacent to leg to spudcan connections;

— strength of the holding system. Hull strength and jackhouse to deck connections are considered to be covered by classification unless special circumstances apply;

— overturning stability and spudcan sliding;

— spudcan strength and foundation bearing capacity.

Checks shall be performed for a range of sea state directions to determine the maximum limit state utilizations.

See also Clauses 9, 12 and 13.

10.6 Fatigue analysis

A fatigue analysis is normally undertaken during the jack-up design phase. For jack-up operations of shorter duration than the RCS special survey period, fatigue analysis is not required provided that an RCS structural integrity regime, or equivalent, is in place. For jack-up operations of longer duration fatigue shall be considered, see Clause 11.

10.7 Earthquake analysis

This subclause addresses analysis of a jack-up using exposure level L1 earthquake data, see 5.5.5.

An earthquake assessment shall be performed for sites where the ISO 19901-2 seismic zone is 2 or above. It is not necessary to perform an earthquake assessment for seismic zone 0. For seismic zone 1, an earthquake assessment should be considered when any of the following conditions applies:

— sites with the potential for cyclic mobility (e.g. liquefaction) (ISO 19901-2 site class F);

— sites with the potential for unacceptable additional leg penetrations if the preload reactions are exceeded (settlement limits can be reduced when operating adjacent to other structures);

— jack-ups where the ratio between the individual leg preload reaction at the spudcan and the maximum still water operating reaction at the spudcan is less than 1,25.

For the relevant zone 1 or higher zones the structure may be assessed using an ELE screening assessment to ULS criteria. The ELE screening earthquake actions shall be derived from the uniform hazard spectrum for a return period of 1 000 years. Guidance on 1 000 year earthquake response spectrum criteria can be found in ISO 19901-2. In this kind of earthquake, the jack-up should sustain little or no damage.

If the jack-up does not satisfy this 1 000 year ELE screening to ULS assessment criteria or the ELE screening assessment has not been performed, the alternative assessment methods (see 10.10) in combination with ISO 19901-2 shall be used to evaluate conformity with the earthquake performance requirements. In this case, the jack-up is acceptable if the assessment demonstrates that structural failures causing loss of life and/or major environmental damage do not occur under any of the earthquake events considered although, in some cases, considerable structural damage can be sustained.

NOTE 1 The dynamic effects of the soil column are not specifically addressed in the screening assessment, however they are included implicitly in the response spectra amplification coefficients. In the alternative assessment approach, the non-linear soil behaviour and its effect on the soil dynamics can be included in the Site Response Analysis or a more detailed soil representation.

The effect of the earthquake on the cantilever hold-down and other critical parts of the jack-up shall be considered.

Earthquake actions shall include accelerations due to the fundamental modes of vibration as well as higher frequency modes associated with the legs above and below the hull, and significant drilling facilities. In addition, the local actions from soil movement on the spudcans and the legs should be considered, where relevant. The associated inertial actions on all significant masses shall be taken into account.

Partial action factors for earthquake assessments are given in 8.8.1.1 and 8.8.1.4.

Since it is not possible to ready the jack-up for an earthquake, it is important to consider reasonable mass and operating configurations.

For earthquake assessments, the spudcan internal entrapped mass shall be included in the mass model and the spudcan added mass (surrounding water and/or soil) shall be included where significant.

NOTE 2 A low mass tends to lead to a shorter natural period and, hence, greater amplification. A higher mass results in a longer period but can be associated with greater lateral forces depending on the reduction in the transverse accelerations in combination with the increased mass.

The assessment model shall include a realistic range of spudcan-soil modelling that encompasses the uncertainties in foundation stiffness and capacities. For earthquake excitation, foundation fixity tends to increase the inertial response and shall be considered - a pinned spudcan model, in general, produces an unconservative representation of the earthquake demand on the jack-up. Where the penetration predictions vary significantly, the range shall be considered.

Spudcan settlement resulting from earthquake excitation shall be considered. Differential settlements can have the most serious consequences.

At sites where cohesionless soil conditions dominate, the possibility of earthquake-induced soil cyclic mobility shall be considered (see 9.4.6).

10.8 Ice

10.8.1 General

Jack-ups operating in arctic and cold regions shall conform with the relevant clauses of this document and ISO 19906, as appropriate. Arctic and cold regions are taken to be those areas that can be affected by sea ice, icebergs and icing conditions.

When the annual probability of ice interaction with jack-up is less than 10^{-4}, then ice actions need not be assessed.

When 10^{-4} < (annual probability of ice interaction with jack-up) < 10^{-2} an ALS ice-assessment shall be undertaken (see 10.8.3).

When the annual probability of ice interaction with jack-up is greater than 10^{-2}, ULS and ALS ice-assessments shall be undertaken (see 10.8.2 and 10.8.3). The annual probability of ice interaction can be demonstrated to be reduced through analysis and use of ice management, removal (moving off) and seasonality (see 19906:2019, 8.2.7, ISO 35104).

See A.10.8.1.1 for examples of different operating area types.

10.8.2 ULS

When undertaking ULS ice assessments in extreme assessment situations, see 5.3 a):

— The extreme level wind, wave and current return period shall be taken from this document.

— The extreme wind, wave and current action factor shall be taken from this document.

— The gravity action factors shall be taken from this document.

— The extreme level ice probability of exceedance shall be taken from ISO 19906:2019, 7.2.2.3.

— The extreme level ice action factors shall be taken from ISO 19906:2019, Table 7-3.

— In the absence of a joint probability analysis, combination factors shall be taken from ISO 19906:2019, Table 7-2.

10.8.3 ALS

When undertaking ALS ice assessments in abnormal assessment situations:

— The abnormal level wind, wave and current probability of exceedance shall be taken from ISO 19906.

— The abnormal level wind, wave and current action factor shall be taken from ISO 19906.

— The gravity action factors shall be taken from this document.

— The abnormal level ice probability of exceedance shall be taken from ISO 19906:2019, 7.2.2.4.

— The abnormal level ice action factor shall be taken from ISO 19906:2019, Table 7-3.

— In the absence of a joint probability analysis, combination factors shall be taken from ISO 19906:2019, Table 7-2.

10.8.4 Assessments in the area types

A.10.8.4 gives examples of the assessment used in the different area types.

10.8.5 Additional factors for arctic and cold regions

The assessment of operations in arctic and cold regions shall account for factors additional to those addressed for other regions. See A.10.8.5.

10.9 Accidental situations

Accidental situations are not normally addressed as part of an assessment unless specifically required by the jack-up owner, operator or regulator (see also 5.3).

10.10 Alternative analysis methods

10.10.1 Ultimate strength analysis

An ultimate strength analysis is intended to identify the collapse strength of the jack-up structure and foundation under applied actions. For occupied situations, the acceptance criteria are typically set by the regulator. For unoccupied/occupied-evacuated situations, the acceptance criteria shall be agreed between the operator and the jack-up owner. In some areas of the world, the analysis can entail:

- assessing the jack-up for abnormal wave condition to demonstrate survivability (e.g. for a 10 000 year return period in the North Sea);
- scaling the extreme storm actions until failure is predicted to occur, to meet a target reserve strength ratio (e.g. Gulf of Mexico fixed structures; see ISO 19902:2020, 9.10.2);
- performing time-history analyses for the ALE (see ISO 19901-2);
- performing ice ALS analyses.

The uncertainties associated with foundation capacity can be significantly greater than those associated with the ultimate strength of the structure. In performing ultimate strength analyses, it is therefore important to make this distinction and to evaluate both structural and foundation failure modes. Therefore, the following strategy is recommended.

a) Structural or foundation failure should be identified using an analysis based on mean (or best estimates) of structural steel properties and soil properties.

b) Where foundation failure occurs before structural failure, structural failure should be determined assuming a foundation fixity based on upper bound or, if necessary, artificially strong, estimates of soil properties. The foundation displacement due to the foundation failure should be appropriately modelled. This should provide an assessment of the steel structure strength.

Ultimate strength evaluation is used to estimate the most likely collapse strength of a structure with partial resistance factors set to 1,0. Due to the absence of partial resistance factors, an ultimate strength evaluation shall be interpreted and used with care.

10.10.2 Methodology

Methodology for performing an ultimate strength analysis can be found in ISO 19902. The determination of actions and foundation properties shall be in accordance with this document.

11 Long-term applications

11.1 Applicability

When a jack-up is to be operated at one particular site for longer than the special survey period, the site-specific assessment shall be supplemented by the provisions of Clause 11 and the requirements of the RCS classing the jack-up.

There can be additional specific requirements of the jack-up owner, operator and regulator related to the long-term application.

11.2 Assessment data

In addition to the data normally required for short-term assessment, further data associated with long-term use are required. These data shall include:

— the duration for which the jack-up is intended to be on site;

— a list of modifications to the jack-up, which affect the time-varying actions, structural resistance or fatigue endurance of structural components;

— the limitations on the ability to re-level the hull and maintain hull elevation, e.g. in connection with supported conductors;

— the deviations from the standard operating and elevated storm mode configurations given in the marine operations manual;

— the metocean data suitable for fatigue assessment, including directionality of wind, waves and current;

— the expected accumulation and vertical distribution of marine growth and relevant mitigation procedures;

— the geotechnical data required for the assessment of long-term operations;

— other data required for fatigue assessment (see 11.3.1).

11.3 Special requirements

11.3.1 Fatigue assessment

The remaining fatigue life of all relevant structural components shall be shown to be adequate for the planned period on site. In the assessment, any fatigue damage contributions from the jack-up's prior service shall be taken into account; historical jack-up and site data shall be requested from the jack-up owner. In view of the inherent uncertainty of fatigue life assessments, a margin of safety shall be applied through a fatigue damage design factor (f_{FD}). See A.11.3.1 for further details.

The partial action factors used for fatigue analysis can be reduced to unity when using S-N curves at mean minus two standard deviations of $\log(N)$.

11.3.2 Weight control

Changes in weight during the long-term operations shall be monitored to ensure conformity with the assessment assumptions. A sufficient allowance for weight growth shall be included in the assessment.

11.3.3 Corrosion protection

Adequate corrosion protection shall be implemented to cover the entire duration on site. Special attention shall be given to corrosion protection in the splash zone.

11.3.4 Marine growth

The assessment shall include the effects of the long-term accumulation of marine growth.

11.3.5 Foundations

The assessment shall include consideration of the potential for and effects of

— settlement under extreme storm actions,

— long-term foundation settlement,

— seabed subsidence, e.g. due to reservoir depletion,

— scour, and

— seabed mobility.

11.4 Survey requirements

Surveys are required to ensure that the integrity of the jack-up is maintained during the long-term application. As a minimum, the jack-up owner shall develop a plan that includes the following surveys:

a) a special survey prior to deployment on site;

b) project specific surveys in accordance with an in-service inspection programme (PSIIP).

The PSIIP required for long-term operations shall be developed based on:

— RCS requirements;

— the jack-up's prior operating and inspection history;

— the assessment results for the expected operations.

Sea floor surveys shall be included in the PSIIP for sites where scour and/or seabed mobility are known to occur.

If changes to the initially planned duration are proposed by the operator, the jack-up owner should document that the jack-up has sufficient remaining fatigue life, and approval is obtained from the RCS and regulator.

12 Structural strength

12.1 Applicability

12.1.1 General

This clause provides the basis for the determination of the structural strength of truss type legs. Limited guidance is given for other leg types. The strength of the fixation system and/or the elevating system and the strength of the spudcan are normally provided by the manufacturer.

Formulae for the required strength checks are given in this clause, which result in structural strength utilizations in accordance with Clause 13.

A suitable method for carrying out the required calculations is given in A.12. The resistance factors given in Annex B are specifically tied to the calculation methods presented in A.12 and shall be re-calibrated if other methods are used.

RCS requirements cover the design, construction, and periodic survey of the jack-up and address issues, such as material properties, fabrication tolerances, welding, construction details and parts of the jack-up other than the legs (e.g. jackhouse and hull structure), which are not normally addressed in a site-specific assessment. For example, when the forces within the fixation system are within the limits set by the manufacturer and are approved by the RCS, no additional assessment is required of the hull and jackhouse. Similarly, if the foundation's vertical and rotational reactions on the spudcan are within the structural limits set by the manufacturer, it is not necessary to check the strength of the leg to spudcan connection.

In this clause, and the corresponding A.12, values for strength, capacity, properties, modulus and geometry are representative values unless indicated otherwise.

12.1.2 Truss type legs

The requirements set out in Clause 12 relate to chords and braces of truss type legs. Weld sizes, gusset plates, the strength of joints, etc., are covered by RCS requirements, and should not control the overall structural integrity. Chords and braces are covered in 12.2 to 12.6.

12.1.3 Other leg types

Some of the checks included in Clause 12 are applicable to either tubular or box-type legs, but for these configurations, Clause 12 should be supplemented with other documents to address stiffened sections, e.g. American Petroleum Institute references API Bulletin 2U (2004)[16] and API Bulletin 2V (2004)[17] or DNV-RP-C202 (DNV 2021b)[58] and DNV-CG-0128 (DNV 2021a)[57].

12.1.4 Fixation system and/or elevating system

The factored representative ultimate strength shall be used for the strength assessment. The strength of the fixation system and/or the elevating system is normally supplied by the manufacturer. The manufacturer's data is not necessarily the unfactored representative ultimate strength of the system(s) but can be a working stress limit value. Data can be given separately for the vertical and horizontal directions. If no representative ultimate strength data are given, or cannot be inferred, then representative ultimate strengths shall be determined through rational analysis.

NOTE An example of a rational approach to determining the ultimate strength is to multiply the allowable rated capacity by 1,15.

12.1.5 Spudcan strength including connection to the leg

The factored representative ultimate strength of the spudcan and the spudcan to leg connection shall be used for the strength assessment. The strength of the spudcan and the spudcan to leg connection is normally supplied by the manufacturer for all applicable vertical and horizontal forces, and for moments about the horizontal axes. The manufacturer's data are not necessarily the unfactored representative ultimate strengths but can be working stress limit values. If no representative ultimate strength data are given, or cannot be inferred, then representative ultimate strengths shall be determined through rational analysis.

NOTE An example of a rational approach to determining the ultimate strength is to multiply the allowable rated capacity by 1,15.

12.1.6 Overview of the assessment procedure

The basic approach consists of the determination of

— classification of member cross-sections (see 12.2),

— section properties of non-circular prismatic members (see 12.3),

— Euler amplification of member forces (if not included within the structural analysis) (see 12.4),

— strength of lattice leg members [tubular members (see 12.5), and prismatic members in truss type legs (see 12.6)], and

— strength of joints (see 12.7).

12.2 Classification of member cross-sections

12.2.1 Member types

The methodology used to classify member cross-sections is different for circular cross-sections of tubular members and for all other cross-sections of prismatic members. Longitudinally reinforced tubulars and tubulars with pin-holes, cut-outs, etc., shall be considered to be non-circular prismatic members.

12.2.2 Material yield strength

The material yield strength used in the member classification and the calculation of member strengths shall correspond to the value at 0,2 % strain offset from the initial linear stress-strain behaviour. A lesser value shall be used when the material does not exhibit sufficient work-hardening.

12.2.3 Classification definitions

The strength of a steel cross-section is affected by its potential to suffer local buckling when subjected to compression due to a bending moment or an axial force, or a combination thereof. By classifying cross-sections, the requirement to explicitly calculate local buckling strength is avoided.

For non-circular prismatic members, the components and cross-sections are classified as plastic, compact, non-compact (or semi-compact) and slender, in order of decreasing strength. When a cross-section is composed of components of different classes, it shall be classified in accordance with the class of its component(s) with the lowest strength in compression. Slender components within a cross-section can be ignored, provided that only the remaining cross-section is used for all aspects of the assessment. The following classification shall be applied.

— Class 1 Plastic: Cross-sections with plastic hinge rotation capacity. Conformity with this classification enables a plastic hinge to develop with sufficient rotation capacity to allow redistribution of moments to occur within the member. All plastic sections are inherently compact.

— Class 2 Compact: Cross-sections with plastic moment capacity. Conformity with this classification enables the full plastic moment capacity of a cross-section to be developed, but local buckling prevents the development of a plastic hinge with sufficient rotation capacity to permit plastic assessment.

— Class 3 Non-compact (or semi-compact): Cross-sections with between full yield moment capacity and plastic moment capacity. Conformity with this classification enables the yield stress to be realized at the extreme compression fibre, but elasto-plastic local buckling prevents development of the full plastic moment capacity.

— Class 4 Slender: Cross-sections that buckle locally before the yield stress can be achieved. A cross-section is classified as slender if any of the compression components of the cross-section does not conform with the limits for non-compact components.

There is no requirement to classify tubular member cross-sections to the same extent as non-circular prismatic member cross-sections other than to identify those tubulars for which plastic hinge rotation capacity is possible (i.e. class 1). This is because the formulae for tubular member cross-sections presented in A.12.5 account for local buckling, whether plastic or elastic.

12.3 Section properties of non-circular prismatic members

12.3.1 General

The requirements in 12.3 apply to rolled and welded non-circular prismatic members comprising one or more components, such as can be found in a chord section of a jack-up leg. Their cross-sectional properties shall be determined as described in 12.3.

Cross-sectional properties of tubular members are included within the determination of their strength and addressed in 12.5.

12.3.2 Plastic and compact sections

For class 1 plastic and class 2 compact sections, section properties can be determined assuming fully plastic properties.

Where elastic section properties are determined for class 1 and 2 sections instead of plastic section properties, these can be based on a fully effective cross-section and shall then be treated as for class 3 sections.

12.3.3 Semi-compact sections

Section properties for class 3 semi-compact sections shall be based on elastic properties assuming fully effective cross-sections. When considering a cross-section comprised of components having different yield strengths, the critical stress locations shall be evaluated as these do not necessarily coincide with the minimum section modulus or the principal axes.

The strength check is based on an interpolation between class 2 plastic capacity and class 3 elastic capacity.

NOTE The critical stress locations are typically at the edges of the components and are a function of the member forces, the yield strength of the component and its position within the cross-section of the member.

12.3.4 Slender sections

Cross-section properties for class 4 slender sections shall be determined using elastic principles. When the stress across the entire section is tensile, the full section may be used. If any part of the section is in compression, the sectional properties shall be reduced as required based on effective sections (see A.12.3.5).

12.3.5 Cross-section properties for the assessment

The nomenclature and selection of variables for use in the assessment of members are summarized in A.12.3.5.

12.4 Effects of axial force on bending moment

The moment resulting from the eccentricity between the elastic and plastic centroids of class 1, 2 and 3 sections shall be included in the assessment moment; this can occur in sections that include components of differing yield strengths. Similarly, for class 4 sections, there is an eccentricity between the full elastic centroid that is used in the structural response analysis and the centroid of the reduced section that is used in the member strength check. This moment correction shall be included for members in both tension and compression.

Euler moment amplification, or p-δ effects, shall be included for members in axial compression. When p-δ effects are not included in the structural response analysis, they shall be included in the strength checks. The effective length factors (K)and moment reduction factors (C_{mr}) for use in strength checks are listed in Table A.12.4-1. Alternatively, they can be determined using a rational analysis that includes joint flexibility and side-sway.

It is mentioned that, traditionally, the effects of Euler amplification are included in the strength checks. However, some analysis results implicitly include the effects of Euler amplification. The assessment should include the effects of both the global large displacement effects (P-Δ) and the local member moment amplification (p-δ). Large displacement effects (P-Δ) are addressed in Clause 8.

12.5 Strength of tubular members

The strength of tubular members shall be checked for combined axial forces and bending, and for shear and torsional shear. The partial factors for the checks required by this subclause are given in Annex B.

The requirements given in 12.5 ignore the effects of hydrostatic pressure. The validity of this assumption shall be checked for all sealed tubular sections (see e.g. Table A.12.5-1).

12.6 Strength of non-circular prismatic members

The strength of non-circular prismatic members shall be checked for combined axial forces and bending, and for shear and torsional shear. The partial factors for the checks required by this subclause are given in Annex B.

The requirements given in 12.6 ignore the effects of hydrostatic pressure. The validity of this assumption shall be checked for all sealed non-circular prismatic members (see e.g. Figure A.12.6-1 and Table A.12.5-1).

12.7 Assessment of joints

Joint strength is normally addressed by the RCS for the metocean conditions given in the operations manual. If the assessor has concerns that the site conditions lead to joint loads that exceed those assessed by the RCS, joint strength shall be assessed.

13 Acceptance criteria

13.1 Applicability

13.1.1 General

This clause defines the criteria for checking the acceptability of a jack-up for operation at a specific site for the various limit states.

The partial action and resistance factors set out in the acceptance criteria have been developed in conjunction with the analysis methodology set out in the rest of this document and are valid only if used with this methodology. The factors do not necessarily provide adequate reliability if used with other methodologies.

The criteria for checking the acceptability of a jack-up include consideration of the following issues:

- structural strength of legs, spudcan, and holding system (see 13.3, 13.4, and 13.5, respectively);
- hull elevation (see 13.6);
- leg length reserve (see 13.7);
- overturning stability (see 13.8);
- foundation integrity including preload, foundation capacity, sliding displacement, settlement resulting from exceedance of the capacity envelope (see 13.9);
- interaction with adjacent infrastructure (see 13.10);
- temperature (see 13.11).

The assessment checks for structural strength, overturning stability and foundation integrity for each limit state and assessment situation are based on a utilization parameter as described in 13.2.

13.1.2 Ultimate limit states

The assessment of the ultimate limit states (ULS) shall ensure that the acceptance criteria are not exceeded in any of the applicable assessment situations; see 5.1, 5.3 and 5.4.

The integrity of the foundation is central to the site-specific assessment of a jack-up.

Areas on jack-ups that are often critical with regard to structural strength are the legs at the lower guides, the legs between guides, the pinions and/or rack teeth, the fixation system and/or fixation system supports (if fixation system is fitted) and the leg to spudcan connection. Where there is a degree of foundation fixity, the lower parts of the leg shall be checked assuming an upper bound fixity value. Foundation fixity may be included in the evaluation of the upper leg when justified by an applicable and detailed foundation study.

Conformity in whole or in part can also be demonstrated through comparison with prior assessments conducted in accordance with the provisions of this document.

13.1.3 Serviceability and accidental limit states

Serviceability limit states and accidental limit states are discussed in 5.3.

13.1.4 Fatigue limit states

For jack-up operations with a duration less than the RCS special survey period, a fatigue analysis is not required, provided that structural integrity is maintained through an appropriate programme of inspection. For long-term applications, fatigue shall be considered in accordance with Clause 11.

NOTE The special survey period is normally between five years and eight years.

13.2 General formulation of the assessment check

The assessment shall follow a partial safety factor format. The partial action factors shall be applied to actions, not the action effects. The partial resistance factors shall be applied to representative foundation capacities and structural strengths. When undertaking a stochastic time domain procedure that incorporates fully non-linear foundation responses, the MPME utilizations shall be calculated using the procedure set out in 10.5.3.

The utilization for each limit state and assessment situation shall satisfy the requirement of Formula (13.2-1):

$$U \leq 1{,}0 \qquad (13.2\text{-}1)$$

where U is the utilization to one significant decimal place.

For assessments where the relevant action effect can be expressed by a single response, U is of the general form given in Formula (13-2-2):

$$U = \frac{A_{F_\mathrm{d}}}{R} \qquad (13.2\text{-}2)$$

where

A_{F_d} is the action effect due to factored actions

R is the factored resistance

For members and foundations subjected to combined forces, the internal force pattern and the resistances combine into a single interaction formula, e.g. combined axial and bending, see A.12.5.3.2 and A.12.6.3. If the interaction formula governing the assessment check is, or can be, reduced to an inequality of the form $U \leq 1{,}0$, then the utilization is equal to U. For assessments where the resistance is given by the yield interaction surface (for foundations) or the plastic interaction surface (for strength of non-circular prismatic members) the utilization is of the general form given in Formula (13-2-3):

$$U = \frac{L_1}{L_2} \qquad (13\text{-}2\text{-}3)$$

where

L_1 is the length of the vector from a specified origin to the factored action effect

L_2 is the length of the vector from the origin specified for L_1 to the factored interaction surface

Factored actions shall be determined in accordance with the assessment load case F_d in 8.8.

Action effects shall be determined in accordance with the requirements of Clauses 9, 10 and 12. Associated guidance is given in A.9, A.10 and A.12. The particular form of the utilization formula is determined by the foundation and strength checks formulated in these clauses.

Annex B summarizes the clause(s)/subclauses(s) in this document where the applicable calculation methodology and the associated assessment check(s) can be found, and lists the values of the partial action and resistance factors that shall be used.

NOTE Normally, both partial action and partial resistance factors are greater than unity: actions are multiplied by partial action factors and resistances are divided by partial resistance factors.

13.3 Leg strength assessment

Formulae (13.2-2) or (13.2-3), as applicable, shall be used to assess the utilization of the leg structure. The methodology for undertaking checks on the strength of members is described in Clause 12, together with the associated resistance factors.

13.4 Holding system strength assessment

The forces on the holding system due to factored actions, for any of the applicable assessment situations, shall be checked against the factored representative value of ultimate strength. A partial resistance factor for holding system strength of $\gamma_{R,H} = 1{,}15$ shall be used.

13.5 Spudcan strength assessment

The forces on the top and bottom of the spudcan due to factored actions, for any of the applicable assessment situations, shall be checked against the factored representative value of ultimate strength. A partial resistance factor for spudcan strength of $\gamma_{R,S} = 1{,}15$ shall be used. Care should be taken when using calculated foundation capacities (e.g. see E.4.9) because the forces can be higher than used in the manufacturer's design case.

NOTE 1 This check addresses issues such as: spudcan overburden (at maximum penetration); spudcan strength (over the range of predicted penetration); and eccentric spudcan support (e.g. due to foundation fixity, sloping seabed or existing spudcan footprints).

NOTE 2 The spudcan strength checks are unlikely to be critical unless the assessment vertical seabed reaction exceeds the maximum design preload reaction.

13.6 Hull elevation assessment

A hull elevation resulting in at least 1,5 m clearance between the assessment return period extreme wave crest elevation and the underside of the hull shall be provided (see 6.4). The extreme wave crest elevation is normally determined from the extreme still water level (SWL) in A.6.4.4 and the wave crest elevation above SWL in A.6.4.2.4.

In some areas of the world an abnormal wave crest elevation (see A.6.4.2.4) that can affect the global response, can be greater than the extreme wave crest elevation plus 1,5 m. The hull elevation shall be sufficient to clear this abnormal wave crest elevation. Where appropriate metocean databases and reliability models exist, the abnormal wave crest elevation can be determined accounting for the joint probability of tide, surge and crest elevation.

The hull elevation shall account for any settlement due to the extreme or abnormal storm event.

NOTE 1 Metocean studies after hurricanes Katrina and Rita have suggested that there exist local wave crest enhancements with a small area of effect, (Forristall, 2007)[71]. When calculating the hull elevation for jack-ups, it is not necessary to consider these local effects over and above the abnormal crest elevation since they do not affect the jack-up globally.

NOTE 2 The air gap is defined in ISO 19900 as the clearance between the highest water surface that occurs during the extreme metocean conditions and the lowest exposed part not designed to withstand wave impingement. This differs from the definition historically used by the jack-up industry.

13.7 Leg length reserve assessment

The leg length reserve above the upper guides should account for the uncertainty in the prediction of leg penetration and account for any settlement. The leg length reserve shall be at least 1,5 m. The greater the uncertainty, the larger the leg length reserve that should be available. A larger reserve can also be required due to the following:

— strength limitations of the top bay;

— the increase in the proportion of the leg bending moment carried by the holding system due to the effective reduction in leg stiffness at the upper guide;

— additional settlement due to scour;

— long-term foundation settlement;

— reservoir settlement.

13.8 Overturning stability assessment

Formula (13.2-2) shall be used to assess margin of safety against overturning of the jack-up. The utilization shall be calculated as the ratio of overturning moment due to the factored actions, M_{OTM}, and the factored stabilizing moment, $R_{d,OTM}$.

The overturning moment, M_{OTM}, shall be calculated about the overturning axis in the most critical assessment situation using the assessment load case F_d. For independent-leg jack-ups, the overturning axes shall pass through any two or more spudcan reaction points. The reaction points are described in 8.6.2 and A.8.6.2. The factored representative value of the stabilizing moment $R_{d,OTM}$ shall be calculated by Formula (13.8-1):

$$R_{d,OTM} = R_{r,OTM}/\gamma_{R,OTM} \qquad (13.8\text{-}1)$$

where

$R_{r,OTM}$ is the representative value of the stabilizing moment;

$\gamma_{R,OTM}$ is the partial resistance factor for stabilizing moment, $\gamma_{R,OTM} = 1{,}05$.

The representative value of the stabilizing moment, $R_{r,OTM}$, shall be calculated for the same assessment situation and about the same axis as used for the calculation of the overturning moment and shall account for the following contributions:

— large deflection (P-Δ) effects shall be included when computing the overturning utilization;

— the minimum stabilizing moment from the most onerous combination of minimum variable load and position of centre of gravity in accordance with 5.3, 5.4.4, 7.4 (see also A.7.4);

— the stabilizing moments provided by a degree of foundation fixity; any stabilizing moments from foundation fixity shall be calculated in accordance with Clause 9, taking account of any reduction of the moment fixity to conform with the yield surface of the foundation.

Large deflection (P-Δ) effects can be included in one of three ways.

a) A reduced stabilizing moment can be calculated from the fixed action with the jack-up at the displaced position resulting from the factored actions.

b) An increased overturning moment can be calculated incorporating the additional overturning of the hull at a displaced condition.

c) The overturning moment can be calculated from the foundation reactions obtained from a large deflection analysis, so the reduction in stabilizing moment due to large deflection effects is implicitly included within the overturning moment.

NOTE The overturning check serves only the purpose of a traditional benchmark; the assessment is governed by the foundation checks.

13.9 Foundation integrity assessment

13.9.1 Foundation capacity check

Formulae (13.2-2) or (13.2-3) as applicable shall be used to assess the foundation. The spudcan reactions due to factored actions shall be checked against the factored capacity in accordance with the requirements of 9.3.6 (see also A.9.3.6).

For a Level 1 foundation integrity check, the preload utilization, $U_{S,pl}$, shall be computed and reported (see e.g. A.9.3.6.2). The utilization shall satisfy Formula (13.9-1) or the alternative formulation of Formula (13.9-2):

$$U_{S,pl} = \frac{V_{st} + W_{BF,A}}{V_{Lo} / \gamma_{R,PRE}} \leq 1,0 \qquad (13.9\text{-}1)$$

or

$$U_{S,pl} = \frac{F_V - W_{BF,o} + B_S}{V_{Lo} / \gamma_{R,PRE}} \leq 1,0 \qquad (13.9\text{-}2)$$

where the symbols are as defined in 4.1 and $\gamma_{R,PRE}$ shall be taken as 1,1.

For a Step 2a check with pinned spudcans, the utilization of the vertical and horizontal foundation capacity, $U_{S,vhm}$, shall be determined (see e.g. A.9.3.6.4.1) and shall satisfy Formula (13.9-3):

$$U_{S,vhm} = \frac{L_{b1}}{L_{b2}} \leq 1,0 \qquad (13.9\text{-}3)$$

where

L_{b1} is the length of the vector from origin used for establishing the bearing utilization $(F_H, F_V)_{ORG}$ to the environmental response point (determined from the factored actions) (F_H, F_V) (see e.g. A.9.3.6.4.1).

L_{b2} is the length of the vector from origin used for establishing the bearing utilization $(F_H, F_V)_{ORG}$ and passing through (F_H, F_V) to the factored vertical-horizontal capacity surface $Q_{VH,f}$ (see e.g. A.9.3.6.4.1).

For a Step 2a check, the utilization of the foundation resistance to sliding, $U_{S,pl}$, shall be computed (see e.g. A.9.3.6.4.2) and shall satisfy Formula (13.9-4):

$$U_{S,vhm} = \frac{L_{s1}}{L_{s2}} \leq 1,0 \qquad (13.9\text{-}4)$$

where

L_{s1} is the length of the vector from origin used for establishing the sliding utilization $(F_H, F_V)_{ORG}$ to the environmental response point (determined from the factored actions) (F_H, F_V) (see e.g. A.9.3.6.4.2).

L_{s2} is the length of the vector from origin used for establishing the sliding utilization $(F_{H,} F_{V})_{ORG}$ and passing through $(F_{H,} F_{V})$ to the factored vertical-horizontal capacity surface $Q_{VH,f}$ (see e.g. A.9.3.6.4.2).

For a Step 2b check with a degree of foundation fixity, the conditions of Formulae (13.9-3) and (13.9-4) remain valid; (see e.g. A.9.3.6.5).

In a Step 2c check, using a yield interaction or continuum foundation model, conformity with the foundation yield surface is inherently included and the above utilization checks are generally not performed. However, when sliding is not included in the model, a sliding check shall be undertaken (see e.g. A.9.3.6.4.2) and Formula (13.9-4).

13.10 Displacement check

If the forces on any spudcan due to the assessment load case F_d result in a utilization computed in accordance with 13.9.1 that exceeds 1,0, a further assessment may be performed as discussed in A.9.3.6.6. This assessment shall show that any additional settlements and/or the associated additional structural action effects are within acceptable limits. Furthermore, there shall be no operational limitations on levelling the hull and re-establishing a safe hull elevation, or alternatively safely departing the site.

NOTE A conservative estimate of the allowable settlement can be derived from the hull inclination limit if this is specified in the operations manual.

13.11 Interaction with adjacent infrastructure

The displacement of the jack-up shall not:

— lead to contact or adverse interaction with any adjacent structure;

— exceed practical limitations for continued operations.

13.12 Temperatures

The 50 year lowest mean daily average air and water temperatures shall be in conformity with the limits given in the operating manual.

NOTE The purpose of this check is to ensure that the field temperature is compatible with the material used in the jack-up construction.

Annex A
(informative)

Additional information and guidance

NOTE The clauses/subclauses in this annex provide additional information and guidance on clauses/subclauses in the body of this document. The same numbering system and heading titles have been used for ease in identifying the subclause in the body of this document to which it relates.

A.1 Guidance on scope

Although this document does not address the integrity of well conductors, the Institute for Petroleum Guidelines (2001)[106] provide guidance on their assessment.

A.2 Guidance on normative references

No guidance is offered.

A.3 Guidance on terms and definitions

No guidance is offered.

A.4 Guidance on symbols

A.4.1 Symbols used in A.1

No guidance is offered.

A.4.2 Symbols used in A.2

No guidance is offered.

A.4.3 Symbols used in A.3

No guidance is offered.

A.4.4 Symbols used in A.4

No guidance is offered.

A.4.5 Symbols used in A.5

No guidance is offered.

A.4.6 Symbols used in A.6

See 4.1.2

A.4.7 Symbols used in A.7

See 4.1.3

A.4.8 Symbols used in A.8

See 4.1.4

A.4.9 Symbols used in A.9

See 4.1.5

A.4.10 Symbols used in A.10

See 4.1.6

A.4.11 Symbols used in A.11

See 4.1.7

A.4.12 Symbols used in A.12

See 4.1.8

A.5 Guidance on overall considerations

No guidance is offered.

A.6 Guidance on data assembled for each site

A.6.1 Scope

No guidance is offered.

A.6.2 Jack-up data

No guidance is offered.

A.6.3 Site data

No guidance is offered.

A.6.4 Metocean data

A.6.4.1 General

The jack-up should be assessed for the extreme storm event (ULS assessment). For occupied jack-ups (category S1), the 50 year return period independent extremes should be used. Alternatively, 100 year joint probability metocean data may be used. The action factors for these two alternatives differ.

If the jack-up life safety category is occupied-evacuated, it is assumed that reliable forecasting of the extreme storm event is feasible, that evacuation plans are established and documented, and that time and resources are available to safely evacuate all personnel from the jack-up and any adjacent structures that can be affected by failure of the jack-up (see 5.5). Under these conditions, hindcast storm characteristics may be computed based on the threshold time horizon of storm formation relative to the jack-up site. The time horizon is defined as the time required for safe evacuation, and the extreme storm event is derived from the population of storms that can develop and impact the jack-up site within that time horizon.

A sudden hurricane is one that forms locally and, due to speed of formation and proximity to infrastructure at time of formation, might not allow sufficient time to evacuate occupied facilities within the time required by the emergency evacuation plan. The population of storms used to derive the sudden hurricane at a given site can therefore be defined in terms of the time horizon required to evacuate the site. For occupied-evacuated jack-ups utilized in these circumstances, consideration should be given to the use of a 50 year return period "sudden hurricane". An unoccupied jack-up may also be assessed using these criteria.

Partial factors for each of these options are presented in 5.5.2.

Site-specific data, if available, should be used for the assessment as regional data do generally not take account of local variations.

Where the actions due to metocean conditions at the site are directional, the jack-up may be aligned on an advantageous heading subject to practical and infrastructure limitations at the site.

A.6.4.2 Waves

A.6.4.2.1 General

The extreme wave environment should be determined in accordance with A.6.4.2.2 to A.6.4.2.10. It should be based on the three hour storm exposure for the relevant assessment return period (e.g. 50 year independent extremes or 100 year joint probability). The seasonally adjusted wave height may be used when appropriate for the proposed operation. When a fatigue analysis is required (see Clause 11), long-term wave data should be obtained.

The assessor should check the consistency of the wave data provided, giving particular attention to the wave periods and the ratio of H_{max} to H_{srp} and query any apparent inconsistencies with the data provider.

A.6.4.2.2 Extreme wave height

The wave height information for a specific site can be expressed in terms of H_{max}, the individual extreme wave height for the assessment return period, or the significant wave height H_{srp}. The relationship between H_{srp} and H_{max} should be determined accounting for the duration of a storm (three hours minimum) and for the additional probability of other return period storms; see ISO/TR 19905-2:2012, 6.4.2.2. This relationship depends on the regional and site-specific conditions however, in the absence of site-specific information, H_{srp} may usually be determined from H_{max} using the generally accepted relationship for non-cyclonic areas as given in Formula (A.6.4-1):

$$H_{max} = 1{,}86\ H_{srp} \quad \text{(A.6.4-1)}$$

Similarly for cyclonic areas, in the absence of site-specific data, the recommended relationship is as given in Formula (A.6.4-2):

$$H_{max} = 1{,}75\ H_{srp} \quad \text{(A.6.4-2)}$$

The wave action can be computed deterministically (through an individual maximum wave approach) or probabilistically (through a time domain simulation). The two methods are discussed in A.6.4.2.3 and in A.6.4.2.5 to A.6.4.2.8, respectively (see also ISO/TR 19905-2:2012, 6.4.2). The two methods should be used in conjunction with the associated kinematics modelling recommended in A.7.3.

A.6.4.2.3 Deterministic waves

For the calculation of wave actions using a deterministic (regular) wave, it is appropriate to apply a kinematics reduction factor to the horizontal and vertical velocities and accelerations in order to obtain realistic estimates of the actions for the extreme storm event. This factor ensures that both the deterministic (regular) calculation of wave action using a regular wave and the three-hour stochastic simulation produce statistically comparable results (i.e. both target the MPME response in the 50 year extreme storm event). In addition, the factor takes some account of wave spreading and the conservatism of regular wave kinematics. The kinematics reduction factor can be applied by scaling of wave kinematics. Use of wave height reduction is not appropriate and should not be used.

The kinematics reduction factor, κ, to be applied to the kinematics obtained from H_{max} can be determined from Formula (A.6.4-3):

$$\kappa = \phi \qquad \text{(A.6.4-3)}$$

where

ϕ is the directional spreading factor in accordance with ISO 19901-1:2015, A.8.3.2.2, for the site-specific metocean data or for open water conditions; it is based on the latitude Ψ in degrees and the type of storm or region:

for low latitude monsoons with typically $|\Psi| < 15°$ $\phi = 0{,}88$

for tropical cyclones below approximately 40° latitude $\phi = 0{,}87$

for extratropical storms for the range of latitudes $36° < |\Psi| < 72°$ $\phi = 1{,}019\,3 - 0{,}002\,08\,|\Psi|$.

Alternatively, Formulae (A.6.4-4) to (A.6.4-7) can be used; see Hoyle et al. (2009)[101]:

$$\kappa = 0{,}824\phi + 0{,}426\phi^2 - 0{,}043\left(\frac{S_y}{L_w}\right)\phi - 1{,}450\left(\frac{S_y}{L_w}\right)^2\phi - 0{,}800\left(\frac{d_w}{L_w}\right)\phi + \cdots$$

$$\ldots + 0{,}658\left(\frac{d_w}{L_w}\right)^2 - 0{,}640\left(\frac{H_{max}}{d_w}\right) + 1{,}303\left(\frac{H_{max}}{d_w}\right)^2\phi^2 \qquad \text{(A.6.4-4)}$$

and subject to the following:

$$0{,}08 \le \left(\frac{S_y}{L_w}\right) \le 0{,}43 \qquad \text{(A.6.4-5)}$$

$$0{,}14 \le \left(\frac{d_w}{L_w}\right) \le 0{,}76 \qquad \text{(A.6.4-6)}$$

$$0{,}07 \le \left(\frac{H_{max}}{d_w}\right) \le 0{,}58 \qquad \text{(A.6.4-7)}$$

where

S_y is the smallest spacing between the legs of 3-legged jack-ups;

d_w is the water depth;

H_{max} is the maximum wave height;

T_{ass} is the intrinsic wave period associated with H_{max};

L_w is the wave length of the wave with H_{max} and T_{ass} in water depth d_w, according to the periodic wave theory that is being used.

The limiting values $\frac{S_y}{L_w} = 0{,}43$, $\frac{d_w}{L_w} = 0{,}76$ and $\frac{H_{max}}{d_w} = 0{,}07$ may be applied for calculation of κ in Formula (A.6.4-4) in case these bounds are transgressed. In all cases, it is not necessary that κ be greater than ϕ.

The kinematics reduction factor formulation was developed for 3-legged drag-dominated jack-ups. Caution should be exercised if it is applied to other cases. The formulae should not be applied for the low wave conditions that dominate in FLS assessment; such cases are likely to be outside the limits of applicability, where $\kappa = \phi$ can be applied.

In lieu of using the kinematics reduction factor, the effects of wave spreading can be explicitly included in the analysis method, provided that higher frequency interaction effects (e.g. those due to frequency sum terms) are appropriately modelled through the use of second (or higher) order wave theory. Frequency interaction effects introduce additional actions that offset some of the reduction in actions predicted by three-dimensional linear wave theories. See A.7.3.3.3.2.

The wave actions should be determined using an appropriate wave kinematics model in accordance with A.7.3.3.1.

In the analysis, a single value for the intrinsic wave period T_{ass}, expressed in seconds, associated with the maximum wave can be used. The "intrinsic" period of the wave as seen by an observer moving with the current should be used in the derivation of wave kinematics required for action calculations; guidance is given in ISO 19901-1:2015, 8.3. Unless site-specific information indicates otherwise, T_{ass} is normally between the limits as given in Formula (A.6.4-8):

$$3{,}44\sqrt{(H_{srp})} < T_{ass} < 4{,}42\sqrt{(H_{srp})} \quad \text{(A.6.4-8)}$$

where H_{srp} is the return period of the extreme significant wave height, expressed in metres.

A.6.4.2.4 Wave crest elevation

The wave crest elevation used to determine the minimum hull elevation above the extreme still water level in A.6.4.4 can be obtained from the extreme wave height, H_{max} in A.6.4.2.2, and the appropriate deterministic wave theory in A.7.3.3.3.1.

A reasonably foreseeable extreme return period should be used for this calculation, and should be no shorter than 50 years, even if a lower return period is used for other purposes (e.g. the ULS assessment in tropical storm areas).

For some regions, the abnormal wave crest elevation should be calculated based on storm statistics and according to principles described in ISO 19901-1:2015, A.8.7. Examples for the regional application of these principles can be found in Leggett et al (2007)[125], or for general application in DNV-RP-C205 (DNV 2021d)[60].

If a wave height reduction factor is used in a deterministic wave analysis to account for wave spreading and the conservatism of deterministic (regular) wave kinematics (see A.6.4.2.3), it should not be applied in the calculation of the wave crest elevation.

A.6.4.2.5 Wave spectrum

Where the analysis method requires the use of spectral data, the choice of the analytical wave spectrum and associated spectral parameters should reflect the width and shape of the spectra for the site and the significant wave height under consideration. In cases where the fetch and duration of extreme winds are sufficiently long, a fully developed sea results (this is rarely realized except, for example, in areas subject to monsoons). Such conditions can be represented by a Pierson-Moskowitz spectrum. Where the fetch or duration of extreme winds is limited, or in shallow water depths, a JONSWAP spectrum can normally be applied (see A.6.4.2.7).

Further discussions of wave spectra and spectral density functions for the Pierson-Moskowitz, $S_{PM}(\omega)$, and the JONSWAP, $S_{JS}(\omega)$, wave spectra are presented in ISO 19901-1:2015, A.8.3.1.2. The wave spectral density functions expressed as a function of wave frequency, i.e. $S_{\eta\eta}(f)$, can be found in ISO/TR 19905-2:2012, 6.4.2.5.

A.6.4.2.6 Airy wave height correction for stochastic analysis

When Airy wave theory is used for stochastic (random) wave action calculations, see A.7.3.3.3.2, then it is necessary to account for wave asymmetry, which is not included in Airy wave theory. The significant wave height should be increased to capture the largest wave actions at the maximum crest amplitude. The increased significant wave height, H_s, should be determined as a function of the water depth, d_w, expressed in metres, as given in Formula (A.6.4-9):

$$H_s = \left[1 + \left(10H_{srp} / T_{p,i}^2\right)e^{(-d_w/25)}\right]H_{srp} \tag{A.6.4-9}$$

where

d_w is the still, or undisturbed, water depth (positive);

H_{srp} is the return period extreme significant wave height, expressed in metres;

$T_{p,i}$ is the intrinsic modal or peak period of the wave spectrum, and should be used with the wave kinematics model described in A.7.3.3.3.2.

A.6.4.2.7 Peak and mean zero-upcrossing periods

When undertaking a stochastic analysis (either for a one-stage analysis or for determining a DAF for a two-stage analysis), it is necessary to either consider a range of wave periods or a suitable wave spectrum that contains sufficient breadth of the peak to capture the dynamic characteristics. Information on the range of periods to use is given in this sub-clause, however, to avoid the requirement for dynamic analyses with several different wave periods, a practical alternative is to use a two-parameter spectrum, such as Pierson-Moskowitz with $\gamma = 1{,}0$, in combination with the site-specific most probable peak period. When using the relationships in Table A.6.4-1, the value of γ used should be as given by the data provider.

For a given significant wave height, the wave period depends on the significant wave steepness which in extreme seas in deep water often lies within the range 1/20 to 1/16. This leads to the expression for intrinsic mean zero-upcrossing period $T_{z,i}$, related to H_{srp} in metres, given in Formula (A.6.4-10):

$$3{,}2\sqrt{(H_{srp})} < T_{z,i} < 3{,}6\sqrt{(H_{srp})} \tag{A.6.4-10}$$

However, in shallow water the wave steepness can increase to 1/12 or more, leading to an intrinsic mean zero-upcrossing period $T_{z,i}$ as low as $2{,}8\sqrt{(H_{srp})}$. This is because in shallow water the wave height increases and wave length decreases for a given $T_{z,i}$.

When considering a JONSWAP spectrum, the peak enhancement factor γ varies between 1 and 7 with a most probable average value between 2,0 and 3,3. There is no firm relationship between γ, H_s and $T_{p,i}$. Relationships between variables for different γ according to Carter (1982)[40] are given in Table A.6.4-1.

Table A.6.4-1 — Relationship between γ, $T_{z,i}$ and $T_{p,i}$

γ	$T_{p,i}/T_{z,i}$
1	1,406
2	1,339
3	1,295
3,3	1,286
4	1,260
5	1,241
6	1,221
7	1,205

Unless site-specific information indicates otherwise values of γ between 2,0 and 3,3 can be used, selecting the value that produces the largest DAF.

If a JONSWAP spectrum is applied, the response analysis should consider a range of periods associated with H_{srp} based on the most probable value of $T_{p,i}$ plus or minus one standard deviation. However, it should be ensured that the assumptions made in deriving the spectral period parameters are consistent with the values used in the analysis. Alternatively, applicable combinations of wave height and period can be obtained from a scatter diagram determined from site-specific measurements; in this case, specialist advice should be obtained on a suitable spectral form for the site.

For other spectrums the assessor is referred to DNV-RP-C205 (DNV 2021d) for guidance.

A.6.4.2.8 Short-crested stochastic waves

For calculations of stochastic (random) wave actions, the short-crestedness of waves (i.e. the angular distribution of wave energy about the dominant direction) may be taken into account when site-specific information indicates that such effects are applicable. In all cases the potential for increased response due to short-crested waves should be investigated. The effect may be included by means of a directionality function $F(\alpha_w)$, given in Formula (A.6.4-11):

$$S_{\eta\eta}(f,\alpha) = S_{\eta\eta}(f)F(\alpha_w) \qquad \text{(A.6.4-11)}$$

where

α_w is the angle between the direction of elementary wave trains and the dominant direction of the short-crested waves;

$S_{\eta\eta}(f,\alpha_w)$ is the directional short-crested power density spectrum;

$F(\alpha_w)$ is the directionality function.

Directionality functions for extreme and fatigue analyses can be found in ISO 19901-1:2015, A.8.3.2.1, and ISO/TR 19905-2:2012, 6.4.2.8. When referring to the formulations in ISO 19901-1:2015, A.8.3.2.1, swell sea parameter ranges should be used for extreme analysis and wind sea parameter ranges for fatigue analysis.

NOTE If using the approach in ISO 19901-1:2015, A.8.3.2.1, then the directional spreading function D_1 with $n = 8$ gives good agreement with the formulation in ISO/TR 19905-2:2012, 6.4.2.8. For directional spreading function D_2 with $s = 15$ and for directional spreading function D_3 with $\sigma = 0{,}34$ there is good agreement with the formulation in ISO/TR 19905-2:2012, 6.4.2.8.

The modelling of short-crested stochastic waves should not be combined with the wave kinematics factor used in deterministic wave analysis to represent wave spreading and the conservatism of deterministic (regular) wave kinematics; see A.6.4.2.3.

A.6.4.2.9 Maximizing the wave/current response

Where the natural period of the jack-up is such that it can respond dynamically to waves; see A.10.4.1, the maximum dynamic response can be caused by waves or sea states with periods outside the ranges given in A.6.4.2.3 and A.6.4.2.7. Such conditions should also be investigated to ensure that the maximum (dynamic plus quasi-static) response is determined by considering sea states with different combinations of significant wave height and spectral period, or deterministic waves with different combinations of individual wave height and period. Such combinations may be limited to probabilities of exceedance that are equal to or lower than the intended probability level of the assessment.

A.6.4.2.10 Long-term wave data

For fatigue calculations (see 11.3.1), the long-term wave climate is required. For fatigue analysis, the long-term data present the probability of occurrence for each sea state, characterized by wave energy spectra and the associated physical parameters. This can be presented in the form of a significant wave height versus mean zero-upcrossing period scatter diagram or as a table of representative sea states.

A.6.4.3 Current

Current components should be applied in the downwind direction. The extreme wind-driven surface current velocity should be that associated with the assessment return period wind. When directional information regarding other current velocity components is available, the downwind component of the maximum surface flow of the mean spring tidal current and the assessment return period surge current should be added to the wind-driven surface current as indicated below. When appropriate, the currents can be seasonally adjusted. If directional data are not available, the components should be summed algebraically and assumed to be omnidirectional.

A site-specific study should normally define the current velocity components.

The current profile can be defined by a series of velocities at a range of elevations from sea floor to water surface. Unless site-specific data indicates otherwise, and in the absence of other residual currents (such as circulation, eddy currents, slope currents, internal waves, inertial currents, etc.), an appropriate method for computing current profile (see Figure A.6.4-1) is as given in Formulae (A.6.4-12) and (A.6.4-13):

$$V_C = (V_t + V_s)[(d_w + z)/d_w]^{1/7} + V_w\,[(h_{ref} + z)/h_{ref}] \qquad \text{for } |z| \le h_{ref} \qquad \text{(A.6.4-12)}$$

$$V_C = (V_t + V_s)[(d_w + z)/d_w]^{1/7} \qquad \text{for } |z| > h_{ref} \qquad \text{(A.6.4-13)}$$

where

V_C is the current velocity as a function of z;

NOTE A reduction can be applicable according to A.7.3.3.4.

d_w is the water depth

V_t is the downwind component of mean spring tidal current;

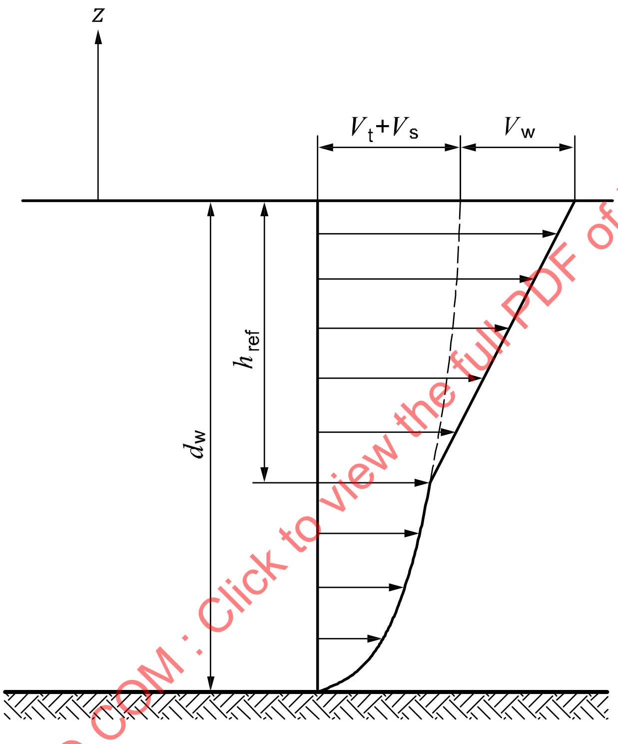
V_s is the downwind component of associated surge current (excluding wind-driven component);

V_w is the wind generated surface current; in the absence of other data, this may conservatively be taken as 2,6 % of the 1 min sustained wind speed at 10 m;

h_{ref} is the reference depth for wind-driven current, in the absence of other data, h_{ref} should be taken as 10 m;

z is the vertical coordinate relative to the SWL under consideration, positive upwards (always negative in the water column).

Alternative formulations are provided in ISO 19901-1:2015, A.9.3. Comparisons of combined current and wave actions in ISO/TR 19905-2:2012, 6.4.3, show that the constant current profile is on the conservative side compared to the power law formulations presented in ISO 19901-1.



Key

d_w water depth
h_{ref} reference depth for wind-driven current
V_s downwind component of surge current
V_t downwind component of tidal current
V_w wind-driven surface current
z vertical coordinate relative to the SWL under consideration, positive upwards (always negative in the water column)

Figure A.6.4-1 — Suggested current profile, adapted from DNV-RP-C104 (DNV 2022b)

In the presence of waves the current profile should be stretched/compressed such that the surface component remains constant. This can be achieved by substituting the elevation as described in A.7.3.3.3.2. Alternative methods can be suitable, however mass continuity methods are not recommended.

The current profile can be changed by wave breaking. In such cases the wind-induced current could be more uniform with depth.

For a fatigue analysis, current can normally be neglected.

A.6.4.4 Water depths

The mean sea level (MSL) is used as the refence level for wind speed and marine growth. The SWLs used for the assessment of the site should be determined and related to LAT. The relationship between LAT and CD is discussed in ISO/TR 19905-2:2012, 6.4.4.

— Different extreme water levels are required for the ULS assessment and hull elevation determination.

  — Unless reliable joint probability data are available, the extreme SWL, expressed as a height above LAT can be taken as follows:

    mean high water spring tidal level+ relevant return period extreme storm surge.

  — When lower water levels are more onerous for action calculations, the minimum SWL expressed as a height above LAT should be taken as follows:

    mean low water spring tidal level + relevant return period negative storm surge.

— When determining the SWL for air gap calculations (safe hull elevation), a reasonably foreseeable extreme return period should be used. This should be no shorter than 50 years, even if a lower return period is used for other purposes (e.g. the ULS assessment in tropical storm areas).

A.6.4.5 Marine growth

Site-specific data should be obtained. In the absence of such data, default values for thickness and distribution are given in A.7.3.2.5.

A.6.4.6 Wind

A.6.4.6.1 General

The wind velocity used for the assessment return period should be the 1 min sustained wind speed, related to a reference level of 10 m above MSL.

The wind velocity profile may be defined by a logarithmic function in accordance with ISO 19901-1, or approximated by a power law (see A.6.4.6.2). A comparison of wind actions shows that the power law profile is slightly more severe than the ISO 19901-1 logarithmic profile, see ISO/TR 19905-2:2012, 6.4.6.1. Typically, the average difference is in the range of 7 % for a 1 min average wind speed of 20 m/s at 10 m above sea level, and 2 % for a 1 min average wind speed of 40 m/s.

Different jack-up configurations (weight, centre of gravity, cantilever position, etc.) may be specified for operating and elevated storm modes. In such cases, the maximum wind velocity considered for the operating mode should not exceed that permitted for the change to the elevated storm mode.

Formulae for the calculation of wind actions are given in A.7.3.4.

A.6.4.6.2 Wind profile

An expression for the vertical profile of the mean wind speed in the form of a power law is given by Formulae (A.6.4-14) and (A.6.4-15):

$$V_Z = V_{\text{ref}}(Z/Z_{\text{ref}})^{1/N_\text{w}} \quad \text{for } Z \geq Z_{\text{ref}} \tag{A.6.4-14}$$

$$V_Z = V_{\text{ref}} \quad \text{for } Z < Z_{\text{ref}} \tag{A.6.4-15}$$

where

V_z is the wind speed at elevation Z above the SWL under consideration;

V_{ref} is the 1 min sustained wind speed at elevation Z_{ref} (normally 10 m);

Z is the elevation above the SWL;

Z_{ref} is the reference elevation above the SWL;

N_W is the inverse exponent of the power law profile; $N_W = 10$ unless site-specific data indicate that an alternative value of N is appropriate.

Alternative profiles may be used when justified by the site-specific data.

A.6.5 Geophysical and geotechnical data

A.6.5.1 Geoscience data

A.6.5.1.1 General

Adequate geophysical and geotechnical information should be available to assess the suitability of the site and the foundation stability. The area covered should be sufficiently large to encompass any stand-off location; normally a 1 km × 1 km square is sufficient. For areas with regional geohazard issues, it is prudent to adopt a larger survey area to quantify the risk of potential geohazards, e.g. mud volcanoes, faults. Aspects that should be investigated are shown in Table A.6.5-1 and are discussed in more detail in the referenced subclauses. The information obtained from the surveys and investigations set out in A.6.5.1.2 to A.6.5.1.5 is required for areas where there is no adequate data available from previous operations. In areas where information is available, the recommendations set out herein may be considered using information obtained from other surveys or activities in the field.

Detailed guidance on geophysical and geotechnical site investigations can also be found in ISO 19901-10 and ISO 19901-8, respectively.

Experience of prior jack-up operations in the same field should be considered, particularly when the previous bearing pressures exceed those for the present operation by an adequate margin.

A.6.5.1.2 Bathymetric survey

An appropriate bathymetric survey should be supplied for an area approximately 1 km square centred on the proposed site. Line spacing of the survey should typically be not greater than 100 m × 250 m over the survey area. Interlining should be performed within an area 200 m × 200 m centred on the proposed site. Interlining should have spacing less than 25 m × 50 m. Such surveys are normally carried out using acoustic reflection systems (e.g. high-resolution multibeam echosounder).

A.6.5.1.3 Sea floor survey

The sea floor should be surveyed using sidescan sonar technique and should be of sufficient quality to identify obstructions and sea floor features and should cover the immediate area (normally a 1 km square) around the intended site. The slant range selection should give a minimum of 100 % overlap between adjacent lines. A magnetometer survey should also be undertaken if there are buried pipelines, cables and other metallic debris located on or slightly below the sea floor.

Sufficient information should be obtained to enable safe positioning and removal of the jack-up. Sea floor obstructions, such as pipelines and wellheads, should be identified to sufficient depth to avoid the potential for spudcan interference during both installation on and removal from site. In some cases, a visual inspection should be obtained in addition to the sea floor survey.

Sea floor and debris surveys can become out-of-date, particularly in areas of construction/drilling activity or areas with mobile sediments. Close to existing installations sea floor surveys should, subject to practical considerations, be undertaken immediately prior to the arrival of the jack-up at the site. At sites with no existing surface or subsea infrastructure, the validity of existing sea floor surveys should be determined taking account of local conditions.

Table A.6.5-1 — Foundation hazards, methods for identification and prevention/mitigation

Risk	Methods for identification and prevention/mitigation	Subclause
Installation problems	Bathymetric survey Sea floor survey	A.6.5.1.2 A.6.5.1.3
Punch-through	Shallow seismic survey Soil sampling and other geotechnical testing and analysis Modify the spudcans (when punch-through failure is anticipated in advance)	A.6.5.1.4 A.6.5.1.5, A.9.3.6
Settlement/bearing failure	Shallow seismic survey Soil sampling and other geotechnical testing and analysis Ensure adequate jack-up preload capability	A.6.5.1.4 A.6.5.1.5, A.9.3.6 A.9.3.6
Sliding failure	Shallow seismic survey Soil sampling and other geotechnical testing and analysis Increase vertical spudcan reaction Modify the spudcans (when sliding failure is anticipated in advance)	A.6.5.1.4 A.6.5.1.5, A.9.3.6
Scour and deposition	Bathymetric and sea floor survey (identify sand waves) Surface soil samples and sea floor currents Inspect spudcan foundation regularly Install scour protection (gravel bag/artificial seaweed) when anticipated Modify the spudcans (when scour or deposition is anticipated in advance)	A.6.5.1.2 A.6.5.1.3 A.9.4.7
Geohazards (liquefaction-induced lateral flow, mudslides, mud volcanoes etc)	Sea floor survey Shallow seismic survey Soil sampling and other geotechnical testing and analysis	A.6.5.1.3 A.6.5.1.4 A.6.5.1.5
Gas pockets/shallow gas	Shallow seismic survey, complemented by pilot-hole drilling (where applicable) during subsequent geotechnical survey	A.6.5.1.4
Faults	Shallow seismic survey	A.6.5.1.4
Metal or other object, sunken wreck, anchors, pipelines etc.	Magnetometer and sea floor survey	A.6.5.1.3
Local holes (depressions) in sea floor, reefs, pinnacle rocks, non-metallic structures (e.g. grout blanket) or wooden wreck	Sea floor survey Visual inspection	A.6.5.1.3
Leg extraction difficulties	Soil sampling and other geotechnical testing and analysis Consider change in spudcans (when leg extraction difficulty is anticipated in advance) Jetting/Airlifting	A.6.5.1.5, A.9.4.5 A.9.4.5
Eccentric spudcan reactions	Bathymetry, sea floor & shallow seismic surveys Shallow seismic survey (buried channels or footprints) Soil sampling and other geotechnical testing and analysis Seabed modification	A.6.5.1.2, A.6.5.1.3, A.6.5.1.4 A.6.5.1.4 A.6.5.1.5, A.9.4.2
Seabed slope	Bathymetry, sea floor & shallow seismic survey Seabed modification	A.6.5.1.2, A.6.5.1.3, A.6.5.1.4 A.9.4.2
Footprints of previous jack-ups	Evaluate field records Prescribed installation procedures Consider filling/modification of holes as necessary	A.6.5.1.1, A.6.5.1.2, A.6.5.1.3 A.9.4.3 A.9.4.3

A.6.5.1.4 Shallow seismic survey

A shallow seismic survey uses high resolution acoustic reflection techniques to

— determine near surface soil stratigraphy, and

— reveal the presence of shallow gas concentrations and other geohazards.

NOTE Detection of gas pockets/shallow gas by means of shallow seismic survey alone involves large uncertainties, and shallow gas may not be detectable. In such cases, proportional mitigation measures should be considered, including possible additional detection equipment or drilling of a pilot-hole (see ISO 19901-8 and ISO 19901-10).

Due to the qualitative nature of seismic surveys, it is not possible to conduct analytical foundation appraisals based on seismic data alone. The seismic data should be correlated with existing soil boring data in the vicinity and show similar stratigraphy.

A shallow seismic survey should be performed over an approximately 1 km square area centred on the proposed site. Line spacing of the survey should typically be not greater than 100 m × 250 m over the survey area. The survey report should include at least two vertical cross-sections passing through the proposed site showing all the relevant reflectors and allied geological information. The equipment used should be capable of stratigraphic resolution to 0,5 m and thicker to a depth equal to the greater of 30 m or the anticipated spudcan penetration plus 1,5 times the spudcan diameter.

A.6.5.1.5 Geotechnical investigation

A.6.5.1.5.1 General

Site-specific geotechnical investigation and testing are recommended in areas where any of the following apply:

— relevant and appropriate geotechnical data are not available nearby;

— the shallow seismic survey cannot be interpreted with any certainty;

— significant layering of the strata is indicated;

— the site is known to be potentially hazardous.

A.6.5.1.5.2 Geotechnical investigation scope

A geotechnical investigation should comprise a minimum of one borehole to a depth below the sea floor of 30 m or the anticipated spudcan penetration plus 1,5 times the spudcan diameter, whichever is the greater. All the layers should be adequately investigated and the transition zones cored at a sufficient sampling rate.

The number of boreholes should account for the lateral variability of the soil conditions, regional experience and the geophysical investigation. When a single borehole is made, the borehole should be at the centre of the leg pattern. More detailed recommendations from the InSafeJIP (RPS Energy 2010) are presented in Annex D.

Undisturbed soil sampling, in situ testing and laboratory testing should be conducted. Recognized in situ soil testing tools include piezocone penetrometer (CPT/CPTU), vane shear, T-bar and ball penetrometer tests (see ISO 19901-8).

A.6.5.1.5.3 Geotechnical report

The geotechnical information needed for spudcan emplacement and removal should include borehole logs, in situ test records and documentation of all laboratory tests, together with interpreted soil design parameters. An experienced geotechnical engineer should select design parameters suitable for spudcan foundation assessment. For analyses the geotechnical design parameters should include profiles of undrained shear strength and/or effective stress parameters, soil indices (water content, Atterberg limits, grain size, etc.), relative density, submerged unit weight, remoulded shear strength, soil sensitivity, coefficient of consolidation, and the over consolidation ratio (R_{OC}). All laboratory tests should be performed on high quality samples.

Additional geotechnical information should include cyclic or dynamic soil data that consider soil strength deterioration due to cyclic loading conditions including

— soil stiffness,

— shear modulus,

— strain rate effects,

— foundation damping from radiation effects and material hysteretic losses, and

— shear wave velocities (for use in earthquake site response analysis).

Soil information to evaluate spudcan extraction requires the remoulded or residual soil strength that takes account of the in situ soil strength reduction occurring during spudcan emplacement and the time on site.

The design undrained shear strength utilized for bearing capacity analyses recommended in A.9.3 and A.9.4 are best established by combining results from:

— laboratory tests on unconsolidated samples [e.g. unconsolidated undrained (UU) triaxial and miniature vane tests],

— laboratory tests on consolidated samples (e.g. consolidated direct simple shear tests or consolidated triaxial test in compression or extension), and

— in situ tests (e.g. cone penetrometer tests, ball or T-bar penetration tests, in situ vane tests).

Consideration should be given to available site-specific jack-up installation experience when assessing the appropriate shear strength. In the absence of site-specific experience, it is recommended to use the average undrained shear strength, $s_{u,ave}$, in the equations presented in A.9.3 and A.9.4. Soil strength reduction from spudcan disturbance should be considered when utilizing this design profile. In addition, caution should be taken when applying the average undrained shear strength to bearing capacity calculation procedures that were previously developed and/or calibrated with the undrained shear strength obtained solely from UU triaxial compression tests.

If the recommended UU triaxial, direct simple shear, and triaxial extension strength test data is not available, use of a shear strength design profile based on UU triaxial compression tests has been customary. Historically, strength data from high quality 3,0 in.-diameter push samples have been utilized in the customary best practice. In situ test data, soil disturbance assessment utilizing soil sensitivity, and correlation with site-specific spudcan penetration records can be utilized to refine the design profile based predominately on UU triaxial data.

A.6.5.2 Data integration

The results of bathymetric surveys, sea floor surveys, shallow seismic surveys, seabed samples and geotechnical investigations should be integrated to assess the soil conditions at the proposed site. Lateral variations of geotechnical parameters can be assessed from the correlation of the shallow seismic data and the geotechnical information from the borehole logs and/or in situ tests.

A.6.6 Earthquake data

No guidance is offered.

A.6.7 Ice data

No guidance is offered.

A.7 Guidance on actions

A.7.1 Applicability

Clause A.7 presents formulations and methods that can be applied to calculate actions for site-specific assessments.

The wave and current actions are presented for quasi-static and dynamic analyses in A.7.3. Normally a quasi-static, deterministic extreme wave analysis is performed for jack-up site-specific assessments, and the dynamic effects are represented by an inertial loadset. Calculations of actions for stochastic analysis in time domain simulations are also presented. Such analyses are applicable for calculation of inertial loadsets or for the direct calculation of the structural responses including dynamic effects. The hydrodynamic formulations and coefficients are presented together with formulae for detailed and equivalent modelling of leg hydrodynamic actions.

Wind models, flow coefficients for different structural parts and a formulation for the calculation of static wind actions are presented in A.7.3.4.

Guidance on the determination of the functional actions is presented in A.7.4.

A.7.2 General

No guidance is offered.

A.7.3 Metocean actions

A.7.3.1 General

A.7.3.1.1 Load cases

The wave/current actions on the legs and other structures and the wind actions on the hull, legs and other structures should be considered due to either

a) the 50 year return period individual extremes, or

b) the most onerous combinations of the following 100 year joint probability metocean data:

   1) 100 year return period wave, the associated current and associated wind;

   2) 100 year 1 min wind, the associated wave and associated current;

   3) 100 year current and the associated wave and associated wind.

A.7.3.1.2 Methods for the determination of actions

This subclause describes how the actions are developed for determining the jack-up response by one of two alternative methods, deterministic and stochastic.

A deterministic analysis involves developing static metocean actions and an inertial loadset. The inertial loadset can be developed from either an SDOF method or a stochastic assessment of the wave actions to develop a DAF.

A more detailed stochastic time domain analysis procedure implicitly includes inertial actions and can account for non-linearities of the action and foundation interaction.

The action calculation procedure should follow the steps in the applicable column of Table A.7.3-1.

Table A.7.3-1 — Metocean action calculation procedures

<table>
<tr><th>Topic</th><th>Description</th><th>Deterministic analysis</th><th>Stochastic DAF method</th><th>Fully integrated stochastic analysis</th></tr>
<tr><td>Water depth</td><td>Define storm water depth considering LAT, tide and storm surge</td><td colspan="3">A.6.4.4</td></tr>
<tr><td rowspan="3">Current</td><td>Define current velocity and profile.</td><td colspan="3">A.6.4.3</td></tr>
<tr><td>Determine the effective local current profile by multiplying the specified current profile by a factor accounting for interference from the structure on the flow field.</td><td colspan="3">A.7.3.3.4</td></tr>
<tr><td>Determine the current profile above mean water level in the presence of waves by stretching the current profile such that the surface component remains constant.</td><td colspan="3">A.6.4.3</td></tr>
<tr><td rowspan="7">Wave</td><td>Specify wave height and range of associated wave periods.</td><td colspan="3">A.6.4.2.2
A.6.4.2.3</td></tr>
<tr><td>Determine if supplied wave periods are intrinsic or apparent and calculate the other value that has not been supplied</td><td colspan="3">A.7.3.3.5,
ISO 19901-1:2015, 8.4.4 and A.8.4.3</td></tr>
<tr><td>Define the return period significant wave height and corresponding spectral peak period</td><td>not applicable</td><td colspan="2">A.6.4.2.5, A.6.4.2.7</td></tr>
<tr><td>Calculate effective significant wave height as appropriate</td><td>not applicable</td><td colspan="2">A.6.4.2.6</td></tr>
<tr><td>Specify wave spectrum, wave direction and wave spreading function</td><td>not applicable</td><td colspan="2">A.6.4.2.5, A.6.4.2.8</td></tr>
<tr><td>Calculate wave velocities and accelerations by superposition of intrinsic wave components representing the wave spectrum and wave spreading functions</td><td>not applicable</td><td colspan="2">A.7.3.3.3.2</td></tr>
<tr><td>Is deterministic wave subject to cancellation?</td><td colspan="2">A.10.4.2.5</td><td>not applicable</td></tr>
<tr><td rowspan="2">Wave theory</td><td>Determine the two-dimensional wave kinematics from an appropriate wave theory for the specified wave height, storm water depth, and intrinsic wave period</td><td>A.7.3.3.3.1</td><td colspan="2">not applicable</td></tr>
<tr><td>Apply a reduction factor to the wave kinematics</td><td>A.6.4.2.3</td><td colspan="2">not applicable</td></tr>
<tr><td>Scale the environment</td><td>Apply partial factors to wind, wave and current to match factored deterministic actions</td><td colspan="2">not applicable</td><td>A.10.5.3.2</td></tr>
<tr><td>Hydrodynamic modelling</td><td>Establish detailed or equivalent leg models to represent structural members and appurtenances</td><td colspan="3">A.7.3.2.1, A.7.3.2.2, A.7.3.2.3, A.7.3.2.6</td></tr>
</table>

Topic	Description	Deterministic analysis	Stochastic DAF method	Fully integrated stochastic analysis
	Determine drag and inertia coefficients (detailed or equivalent) as functions of member shape, roughness (marine growth), size, and orientation.	A.7.3.2.4, A.7.3.2.5		
	Include the marine growth thickness relevant for the site and duration of the planned operation	A.7.3.2.5		
Wave/current action	Combine local current profile vectorially with the wave kinematics to determine locally incident fluid velocities and accelerations for calculation of wave and current actions by Morison's equation.	A.7.3.3.3.1, A.7.3.3.3.2		
Wind	Define wind speed and wind profile	A.6.4.6		
Wind action	Define shape coefficients and calculate the static wind action.	A.7.3.4		
Functional actions	Define functional actions	A.7.4		
Other actions	Define other actions	A.7.8		
Stochastic DAF	Does natural period coincide with cancellation or reinforcement	not applicable	A.7.3.3.3.3, A.10.4.2.5	not applicable
	Determine DAF stochastically	not applicable	A.10.5.2.2.3, A.10.5.3	not applicable
Method of inclusion of dynamic effects in analysis	Determine DAF either deterministically or stochastically. Represent dynamic effects by an inertial loadset	A.10.5.2.2.2 A.10.5.2.2.3	← follow deterministic analysis	not applicable
	Does natural period coincide with cancellation or reinforcement?	not applicable	not applicable	A.7.3.3.3.3, A.10.4.2.5
Action factors	Apply action factors to the metocean actions and dynamic effects	8.8.1.2	not applicable	8.8.1.3
Load cases	Develop assessment load case by linearly combining the factored metocean actions with the factored functional actions	8.8.1.1, A.10.5.2.2.3	not applicable	8.8.1.1
	Additional load cases if $(T_n/T_p) > 0,9$	A.10.5.2.2.3	not applicable	not applicable

When a fully integrated stochastic analysis is undertaken (see 10.3), partial factors are applied to the metocean parameters instead of the metocean actions, as described in A.10.5.3 and 8.8.1.3. When using stochastic dynamic analyses for the purpose of determining a DAF, no partial action factors are applied; however, in the subsequent deterministic analysis including the inertial loadset based on the stochastic DAF, the action factors described in 8.8.1.2 are applied.

A.7.3.2 Hydrodynamic model

A.7.3.2.1 General

The hydrodynamic modelling of the jack-up leg can be carried out by utilizing "detailed" or "equivalent" techniques. The hydrodynamic properties are then found as described in A.7.3.2.2 to A.7.3.2.4. In all cases, the provisions in the remainder of A.7.3.2.1 should be considered.

The drag properties of some chords represented by the product of the drag coefficient C_D and reference diameter D_i differ for flow in the direction of the wave propagation (in the wave crest) and for flow back in the opposite direction (in the wave trough). Often the combined drag properties of all the chords on a leg gives a total value along a particular axis that is independent of the flow direction. When this is not the case, it is recommended that the effect is included directly in the wave/current action model. Otherwise, where possible, the following is recommended:

a) regular wave deterministic calculations use drag properties appropriate to the flow direction under consideration, noting that the flow direction is that of the combined wave particle motion and current;

b) for random wave analyses, which are solely used to determine dynamic effects for inclusion in a final regular wave deterministic calculation on the basis of item a) above, an average drag property is considered;

c) for random wave analyses from which the final results are obtained directly, the drag property in the direction of wave propagation is used.

Lengths of members are normally taken as the node-to-node distance of the members in order to account for small non-structural items (e.g. anodes, jetting lines of less than 4" nominal diameter); see NOTE below. Large non-structural items, such as raw water pipes and ladders, should be included in the model. Free standing conductor pipes and raw water towers should be considered separately from the leg hydrodynamic model.

For the purpose of this calculation, a node is defined as the point where two member axes intersect. Offsets between terminating members along the axis of the continuous member at the node may be used when calculating the equivalent C_D.

The contribution of the part of the spudcan above the sea floor should be investigated and only excluded from the model if it is shown to be insignificant. In water depths greater than 2,5H_s or where penetrations exceed half the spudcan height, the effect of the spudcan is normally insignificant. Otherwise, hydrodynamic actions should be modelled with hydrodynamic coefficients applicable for large diameter members; see ISO/TR 19905-2:2012, 7.3.2.4 and 7.3.2.5.

On some jack-ups, the lower section of the leg adjacent to the spudcan can be heavily reinforced for towage; this should be explicitly modelled.

For leg structural members, shielding and solidification effects should not normally be applied in calculating wave actions. The current flow is however reduced due to interference from the structure on the flow field, see A.7.3.3.4.

NOTE The solidification effect, which increases the actions from waves due to interference from objects “side by side” in the flow field, is normally not included in the determination of the hydrodynamic coefficients or jack-ups. Jack-ups are usually space frame structures with few parallel members in close proximity so that shielding and solidification effects are usually not important. However, solidification can be important for closely spaced members such as are found in some raw water systems.

Coefficients for individual members with closely attached appurtenances should be calculated by accounting for the combined shape with reference to relevant literature (DNV-RP-C205, 2021d). Model test data may be used for non-circular members, if available. In such cases the effects of roughness, Keulegan-Carpenter and Reynolds number dependence should be considered. The building block methodology described below was developed and calibrated for SNAME Technical and Research Bulletin 5-5A (2002)[170]. Model tests and analytical studies for complete legs are difficult to interpret and are unlikely to give results that are consistent with the methodology used here. This is particularly true for legs in which tubular members contribute significantly to the total drag coefficient because of Reynolds number dependency.

A.7.3.2.2 "Detailed" leg model

All members are modelled with Morison coefficients accounting for member cross-section orientation relative to the flow direction. Members can be lumped together using the corresponding $C_{\mathrm{D}}D_{\mathrm{r}} = \Sigma C_{\mathrm{D}i}D_i$ and $C_{\mathrm{m}}A = \Sigma C_{\mathrm{m}i}\pi D_i^2/4$, accounting for flow direction, as defined in A.7.3.2.4.

A.7.3.2.3 "Equivalent" leg model

The hydrodynamic model of a bay is comprised of one, "equivalent" vertical tubular located at the geometric centre of the actual leg. The corresponding (horizontal) v_{n} , $\dot{u}_{\mathrm{n}}$ and $\ddot{r}_{\mathrm{n}}$ (see A.7.3.3.2) are applied together with equivalent $C_{\mathrm{D}}D = \Sigma C_{\mathrm{De}}D_{\mathrm{e}}$ and $C_{\mathrm{m}}A = \Sigma C_{\mathrm{me}}A_{\mathrm{e}}$, as defined in A.7.3.2.4. The model should be varied with elevation, as necessary, to account for changes in dimensions, marine growth thickness, etc.

When the hydrodynamic properties of a lattice leg are idealized by an "equivalent" model, the properties can be found using the method given below.

The equivalent value of the drag coefficient, C_{De}, times the equivalent diameter, D_{e}, of the bay can be chosen as given in Formula (A.7.3-1):

$$C_{\mathrm{De}}D_e = D_e \Sigma C_{\mathrm{De}i} \tag{A.7.3-1}$$

The equivalent value of the drag coefficient for each member, $C_{\mathrm{De}i}$, is determined as given in Formula (A.7.3-2):

$$C_{\mathrm{De}i} = [\sin^2 \beta_i + \cos^2 \beta_i \sin^2 \alpha_i]^{3/2} C_{\mathrm{D}i} \frac{D_i l_i}{D_{\mathrm{e}} s} \tag{A.7.3-2}$$

where

$C_{\mathrm{D}i}$ is the drag coefficient of an individual member i as defined in A.7.3.2.4;

D_i is the reference diameter of member i (including marine growth as applicable) as defined in A.7.3.2.4;

D_{e} is the equivalent diameter of leg, suggested as $\sqrt{(\Sigma D_i^2 l_i / s}$;

l_i is the length of member i node to node centre;

s is the length of one bay, or part of bay considered;

α_i is the angle between flow direction and member axis projected onto a horizontal plane;

β_i is the angle defining the member inclination from horizontal (see Figure A.7.3-1).

Σ indicates summation over all members in one leg bay.

The above expression for $C_{\mathrm{De}i}$ can be simplified for horizontal and vertical members as given in Formulae (A.7.3-3) and (A.7.3-4):

vertical members (e.g. chords): $$C_{\mathrm{De}i} = C_{\mathrm{D}i}(D_i / D_{\mathrm{e}}) \tag{A.7.3-3}$$

horizontal members: $$C_{\text{De}i} = \sin^3(\alpha_i) C_{\text{D}i} \left(\frac{D_i l_i}{D_e s} \right) \quad \text{(A.7.3-4)}$$

The equivalent value of the inertia coefficient, C_{me}, and the equivalent area, A_e, representing the bay can be determined from the following:

C_{me} is the equivalent inertia coefficient, which may normally be taken as 2,0 when using A_e;

A_e is the equivalent area of leg per unit height, equal to $(\Sigma A_i l_i)/s$;

A_i is the equivalent area of member or gusset, equal to $\pi D_i^2/4$;

D_i is the reference diameter, chosen as defined in A.7.3.2.4.

For a more accurate model, the C_{me} coefficient may be determined as given in Formula (A.7.3-5):

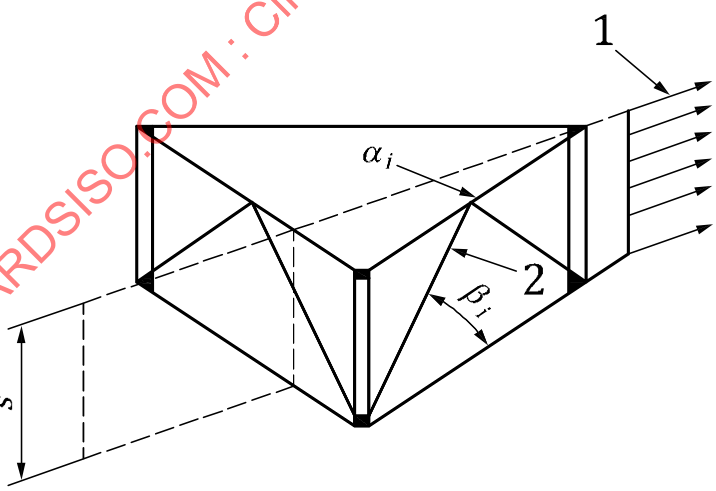
$$C_{me} A_e = A_e \Sigma C_{mei} \quad \text{(A.7.3-5)}$$

where

$$C_{mei} = [1 + (\sin^2\beta_i + \cos^2\beta_i \sin^2\alpha_i)(C_{mi-1})] \left(\frac{A_i l_i}{A_e s} \right) \quad \text{(A.7.3-6)}$$

C_{mi} is the inertia coefficient of an individual member, which is defined in A.7.3.2.4 related to reference dimension D_i.

For dynamic modelling the added mass of fluid per unit height of leg may be determined as $\rho A_i (C_{mi} - 1)$ for a single member or $\rho A_e (C_{me} - 1)$ for the equivalent model, provided that A_e is as defined above.



Key

1 flow direction

2 member *i*

s bay height

α_i angle between flow direction and axis of member *i* projected onto a horizontal plane

β_i angle defining the inclination of member *i* from horizontal

NOTE Based on DNV-RP-C104, (DNV 2022b).

Figure A.7.3-1 — Flow angles appropriate to a lattice leg

A.7.3.2.4 Drag and inertia coefficients

Hydrodynamic coefficients for leg members are given in this subclause. Tubulars, brackets, split tube and triangular chords are considered. Hydrodynamic coefficients including directional dependence are given together with a fixed reference diameter D_i. No other diameter should be used unless the coefficients are scaled accordingly. Unless better information is available for the computation of wave/current actions, the values of drag and inertia coefficients applicable to Morison's equation should be obtained from this subclause.

Recommended values for hydrodynamic coefficients for tubulars with a diameter smaller than 1,5 m are given in Table A.7.3-2, based on the data discussed in the supporting ISO/TR 19905-2:2012, 7.3.2.4.

Table A.7.3-2 — Base hydrodynamic coefficients for tubulars

Surface condition	C_{Di}	C_{mi} for wave load analysis	C_{mi} for earthquake
Smooth	0,65	2,0	2,0
Rough	1,00	1,8	2,0

The smooth values normally apply above MSL + 2 m and the rough values below MSL + 2 m, where MSL is as defined in A.6.4.4. If the jack-up has operated in deeper water and the fouled legs are not cleaned the surface should be taken as rough for wave actions above MSL + 2 m.

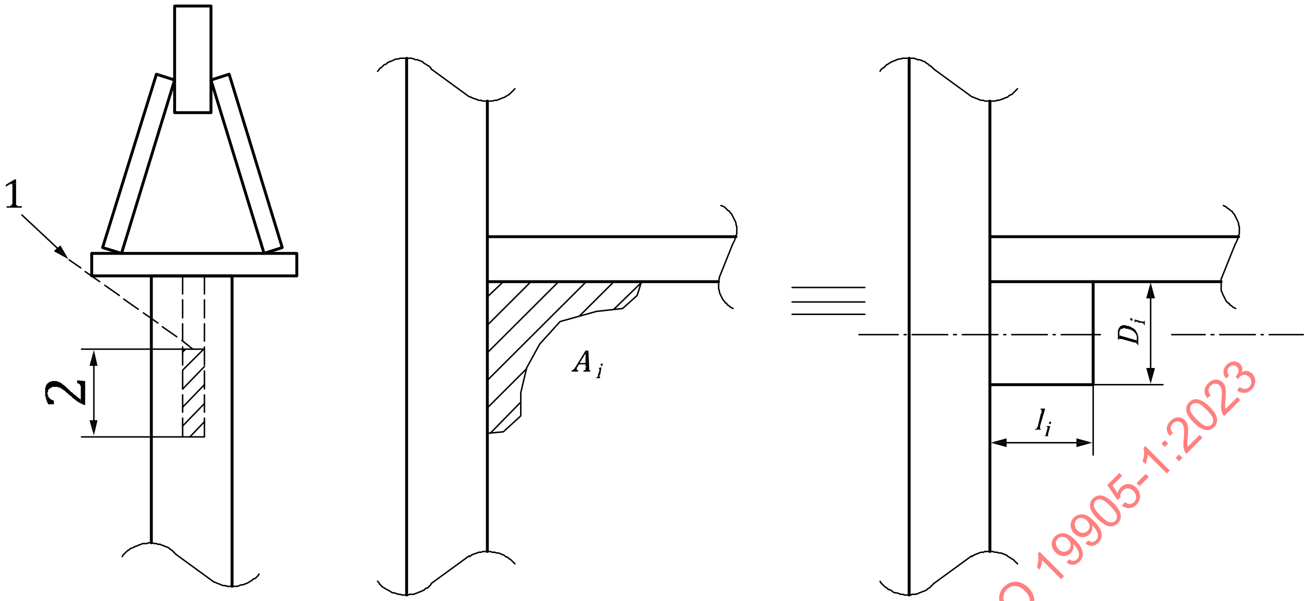
Hydrodynamic coefficients for large diameter members may be calculated in accordance with ISO/TR 19905-2:2012, 7.3.2.4 and 7.3.2.5.

Actions due to gussets should be determined using a drag coefficient as follows:

$$C_{Di} = 2,0$$

applied together with the projected area of the gusset visible in the flow direction, unless model test data show otherwise. This drag coefficient may be applied together with a reference diameter D_i and corresponding length l_i chosen such that their product equals the plane area, $A_i = D_i l_i$ and $D_i = l_i$ (see Figure A.7.3-2). In the equivalent model of A.7.3.2.3 the gussets may be treated as an equivalent horizontal member of length l_i, with its axis in the plane of the gusset. C_{mi} should be taken as 1,0 and marine growth may be ignored.

For non-tubular geometries (e.g. leg chords) the appropriate hydrodynamic coefficients may, in lieu of more detailed information, be taken in accordance with Figure A.7.3-3 or Figure A.7.3-4 and corresponding formulae, as appropriate.



Key

1 flow direction

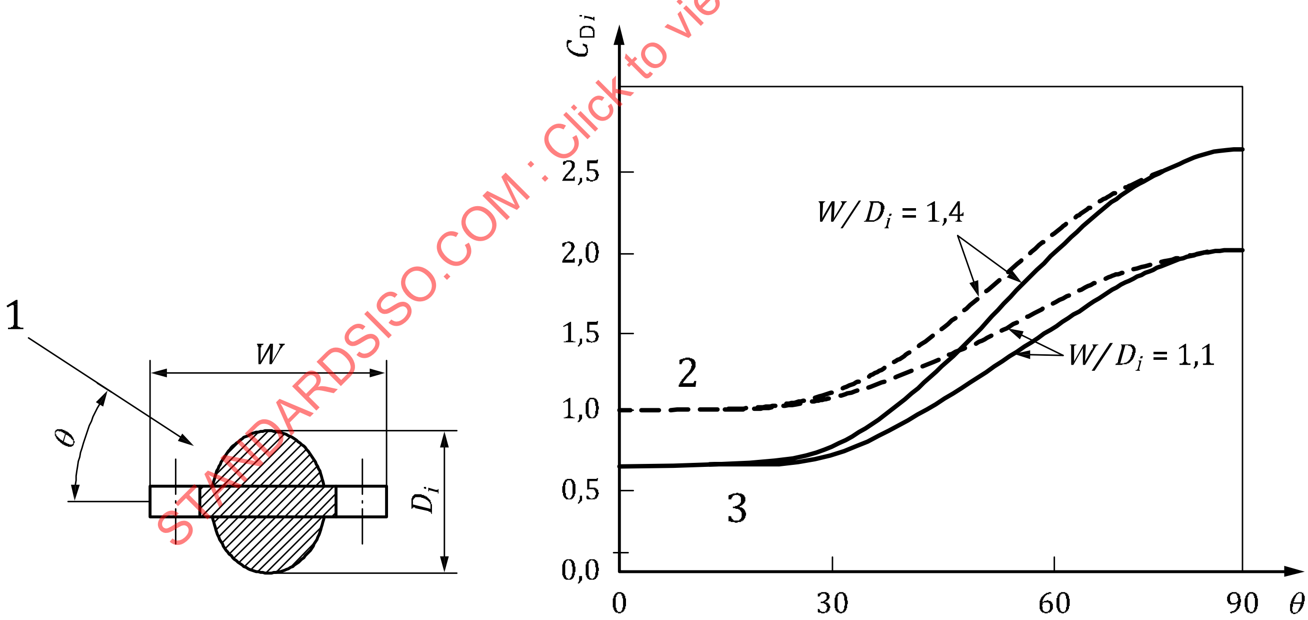
2 visible part of gusset *i*

A_i area of gusset *i*; $A_i = l_i D_i$

D_i reference diameter of gusset *i*

l_i reference length of gusset *i*

Figure A.7.3-2 — Gusset plates: equivalent modelling



Key

1 flow direction

2 rough

3 smooth

C_{Di} drag coefficient for use with D_i

D_i reference dimension of chord *i*

W average width of the rack

θ angle between flow direction and plane of rack (in degrees)

Figure A.7.3-3 — Split tube chord and typical values for C_{Di}

For a split tube chord as shown in Figure A.7.3-3 the drag coefficient C_{Di}, related to the reference dimension $D_i = D + 2t_m$, the diameter of the tubular, including marine growth as in A.7.3.2.3, should be taken from Formula (A.7.3-7):

$$C_{Di} = \begin{cases} C_{Do} & ; \quad 0° < \theta \leq 20° \\ C_{Do} + \left(C_{D1} \dfrac{W}{D_i} - C_{Do} \right) \sin^2 \left[\left(\theta - 20° \right) 9/7 \right] & ; \quad 20° < \theta \leq 90° \end{cases} \tag{A.7.3-7}$$

where

t_m is the marine growth thickness;

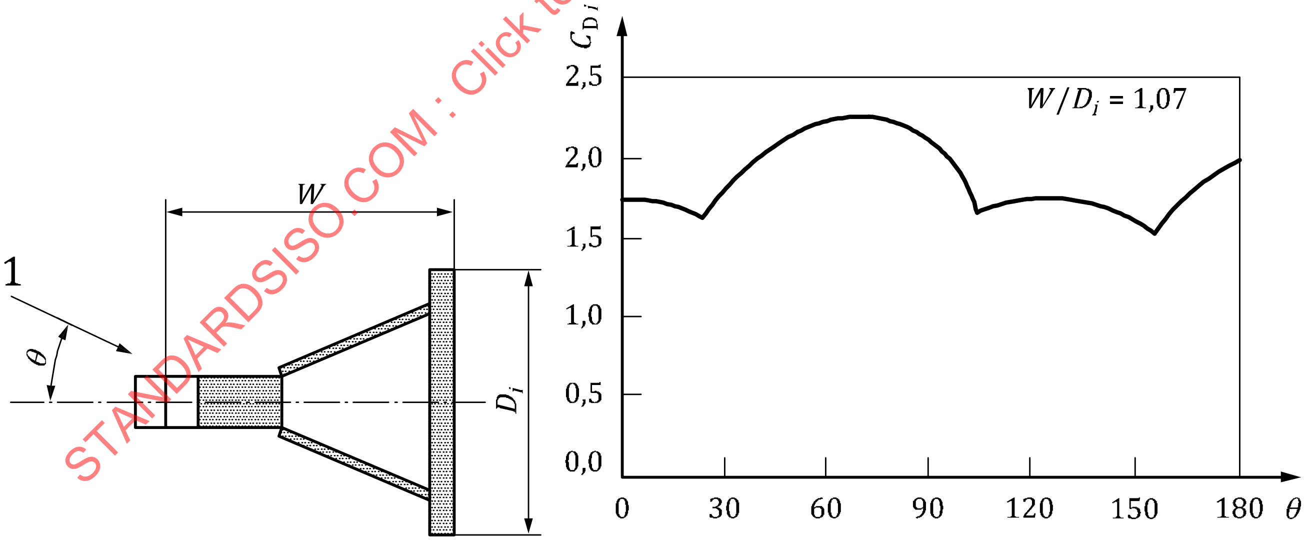
θ is the angle in degrees; see Figure A.7.3-3;

C_{Do} is the drag coefficient for a tubular with appropriate roughness, see Table A.7.3-2;

C_{D1} is the drag coefficient for flow normal to the rack ($\theta = 90°$), related to projected diameter, W. C_{D1} is given by Formula (A.7.3-8):

$$C_{D1} = \begin{cases} 1{,}8 & ; \quad (W/D_i) < 1{,}2 \\ 1{,}4 + (W/D_i)/3 & ; \quad 1{,}2 < (W/D_i) < 1{,}8 \\ 2.0 & ; \quad 1{,}8 < (W/D_i) \end{cases} \tag{A.7.3-8}$$

The inertia coefficient $C_{Mi} = 2{,}0$, related to the equivalent volume $\pi D_i^2/4$ per unit length of member, can be applied to all heading angles and any roughness.



Key

1 flow direction

C_{Di} drag coefficient for use with D_i

D_i reference dimension (height of backplate) of chord *i*

W width of chord to mid-point of rack tooth

θ angle between flow direction and plane of rack (degrees)

Figure A.7.3-4 — Triangular chord and typical values of C_{Di}

For a triangular chord as shown in Figure A.7.3-4, the drag coefficient C_{Di} related to the reference dimension $D_i = D$, the backplate width, should be taken from Formula (A.7.3-9):

$$C_{Di} = C_{Dpr}(\theta)\, D_{pr}(\theta)/D_i \tag{A.7.3-9}$$

where the drag coefficient related to the projected diameter, C_{Dpr}, is determined from Formula (A.7.3-10):

$$C_{Dpr}(\theta) = \begin{cases} 1,70 & ; \quad \theta = 0° \\ 1,95 & ; \quad \theta = 90° \\ 1,40 & ; \quad \theta = 105° \\ 1,65 & ; \quad \theta = 180° - \theta_o \\ 2,00 & ; \quad \theta = 180° \end{cases} \tag{A.7.3-10}$$

Linear interpolation should be applied for intermediate headings. The projected diameter, $D_{pr}(\theta)$, should be determined from Formula (A.7.3-11):

$$D_{pr}(\theta) = \begin{cases} D_i \cos\theta & ; \quad 0 < \theta < \theta_0 \\ W\sin\theta + 0{,}5\, D_i\, |\cos\theta| & ; \quad \theta_0 < \theta < 180 - \theta_0 \\ D_i\, |\cos\theta| & ; \quad 180 - \theta_0 < \theta < 180 \end{cases} \tag{A.7.3-11}$$

The angle θ_o is the angle where half the rackplate is hidden, $\theta_o = \arctan[D_i/(2W)]$.

The inertia coefficient $C_{mi} = 2{,}0$ (as for a flat plate), related to the equivalent volume of $\pi D_i^2/4$ per unit length of member, can be applied for all headings and any roughness.

Shapes, combinations of shapes or closely grouped non-structural items which do not readily fall into the above categories should be assessed from relevant literature (DNV-RP-C205, 2021d) and/or appropriate interpretation of (model) tests. The model tests should consider possible roughness, Keulegan-Carpenter and Reynolds number dependence.

A.7.3.2.5 Marine growth

Some of the influences of marine growth are:

— an increase in the hydrodynamic diameter;

— increases in weight, buoyancy, mass and added mass;

— variation of the hydrodynamic drag coefficient as a function of roughness (see ISO/TR 19905-2).

The thickness and type of marine growth depend on the site and can vary with duration on site, depth and season. Where possible, site-specific or regional data should be used. If such data are not available, all members below MSL + 2 m should be considered to have a marine growth thickness equal to 12,5 mm (i.e. total of 25 mm across the diameter of a tubular member). In some areas of the world, this default thickness can be significantly exceeded.

The nominal sizes of structural members, conductors, risers, and appurtenances should be increased to account for the thickness of pre-existing and new marine growth. Marine growth on the teeth of elevating racks and protruding guided surfaces of chords can normally be ignored.

The marine growth thickness may be ignored if anti-fouling, cleaning or other means are applied. The surface roughness should still be taken into account, see A.7.3.2.4 or ISO/TR 19905-2:2012, A.7.3.2.4.

A.7.3.2.6 Hydrodynamic models for appurtenances

Raw water caissons on the legs and their guides should be included in the hydrodynamic model of the structure.

NOTE The guides for raw water caissons can cause a significant increase in the leg drag load, especially when they are comprised of high drag sections such as I-beams, flat bar, etc.

Depending upon the type and quantity, appurtenances can significantly increase the global wave actions. Appurtenances such as stairways, ladders and jetting lines should be considered for inclusion in the hydrodynamic model of the structure.

Appurtenances are generally modelled by means of increasing the effective diameter and/or hydrodynamic coefficients of a structural member.

A.7.3.3 Wave and current actions

A.7.3.3.1 General

Hydrodynamic actions for deterministic or stochastic analysis should be calculated using the Morison equation in combination with the hydrodynamic model and appropriate wave theories as described in the remainder of A.7.3.3. The wave and current velocities should be combined before they are used in the Morison equation. The intrinsic and apparent wave periods should be used appropriately; see A.7.3.3.5.

A.7.3.3.2 Hydrodynamic actions

Wave and current actions on slender members having cross-sectional dimensions sufficiently small compared with the wave length should be calculated using the Morison equation. The Morison equation is normally applicable providing that

$$L_w > 5D_i \quad \text{(A.7.3-12)}$$

where

L_w is the wave length;

D_i is the reference dimension of member (e.g. tubular diameter).

The Morison equation specifies the action per unit length as the vector sum as given in Formula (A.7.3-13):

$$\Delta F = \Delta F_{\text{drag}} + \Delta F_{\text{inertia}} = 0{,}5\rho D C_D v_n |v_n| + \rho C_m A_{cs} \dot{u}_n - \rho C_A A_{cs} \ddot{r}_n \quad \text{(A.7.3-13)}$$

where the terms of the formula are described as follows.

To obtain the drag action, the appropriate drag coefficient (C_D) should be chosen in combination with a reference diameter, including any increase for marine growth, as described in A.7.3.2.

The Morison drag action formulation is as given in Formula (A.7.3-14):

$$\Delta F_{\text{drag}} = 0{,}5\rho_w C_D D_r v_n |v_n| \quad \text{(A.7.3-14)}$$

where

ΔF_{drag} is the drag action (per unit length) normal to the axis of the member considered in the analysis and in the direction of v_n;

ρ_w is the mass density of water (normally 1 025 kg/m^3);

C_D is the drag coefficient (= C_{Di} or C_{De} from A.7.3);

v_n is the fluid particle velocity resolved normal to the member axis;

D_r is the reference dimension in a plane normal to the fluid velocity v_n. $D_r = D_i$ or D_e from A.7.3.

The fluid particle velocity, v_n, may either be the absolute or relative fluid particle velocity. In a deterministic analysis, the absolute fluid particle velocity is applied. In a stochastic analysis, the fluid particle velocity, v_n, may be taken as given in Formula (A.7.3-15):

$$v_n = u_n + V_{Cn} - \alpha \dot{r}_n \tag{A.7.3-15}$$

where

$u_n + V_{Cn}$ is the combined particle velocity found as the vector sum of the wave particle velocity and the current velocity, normal to the member axis;

$\dot{r}_n$ is the velocity of the considered member, normal to the member axis and in the direction of the combined particle velocity;

α = 0, if an absolute velocity is to be applied, i.e. neglecting the structural velocity;

= 1, if relative velocity is being included. It may be used for stochastic/random wave action analyses only if the following applies:

$u^* T_n / D_i \geq 20$

where

u^* is the particle velocity = $V_C + \pi H_s / T_{z,i}$;

T_n is the first natural period of surge or sway motion;

D_i is the reference diameter of a chord.

NOTE See also A.10.4.3 for relevant damping coefficients depending on α.

To obtain the inertia action, the appropriate inertia coefficient (C_m) should be taken in combination with the cross-sectional area of the geometric profile, including any increase for marine growth, as described in A.7.3.2.3. The Morison's inertia action formulation is as given in Formula (A.7.3-16):

$$\Delta F_{inertia} = \rho\, C_m\, A_{cs}\, \dot{u}_n - \rho\, C_A\, A_{cs}\, \ddot{r}_n \tag{A.7.3-16}$$

where

$\Delta F_{inertia}$ is the inertia action (per unit length) normal to the member axis and in the direction of $\dot{u}_n$;

C_m is the inertia coefficient;

A_{cs} is the cross-sectional area of member (equal to A_i or A_e from A.7.3.2);

$\dot{u}_n$ is the wave particle acceleration normal to member;

C_A is the added mass coefficient, $C_A = C_m - 1$;

$\ddot{r}_n$ is the acceleration of the considered member, normal to the member axis and in the direction of the combined particle acceleration.

The last term in Formula (A.7.3-16) is not included in a deterministic analysis. The term should be included in a stochastic analysis representing the added mass force due to the member acceleration.

$$m_a \ddot{r}_n = \rho C_A A_{cs} \ddot{r}_n \quad \text{(A.7.3-17)}$$

where m_a is the added mass contribution (per unit length) for the member.

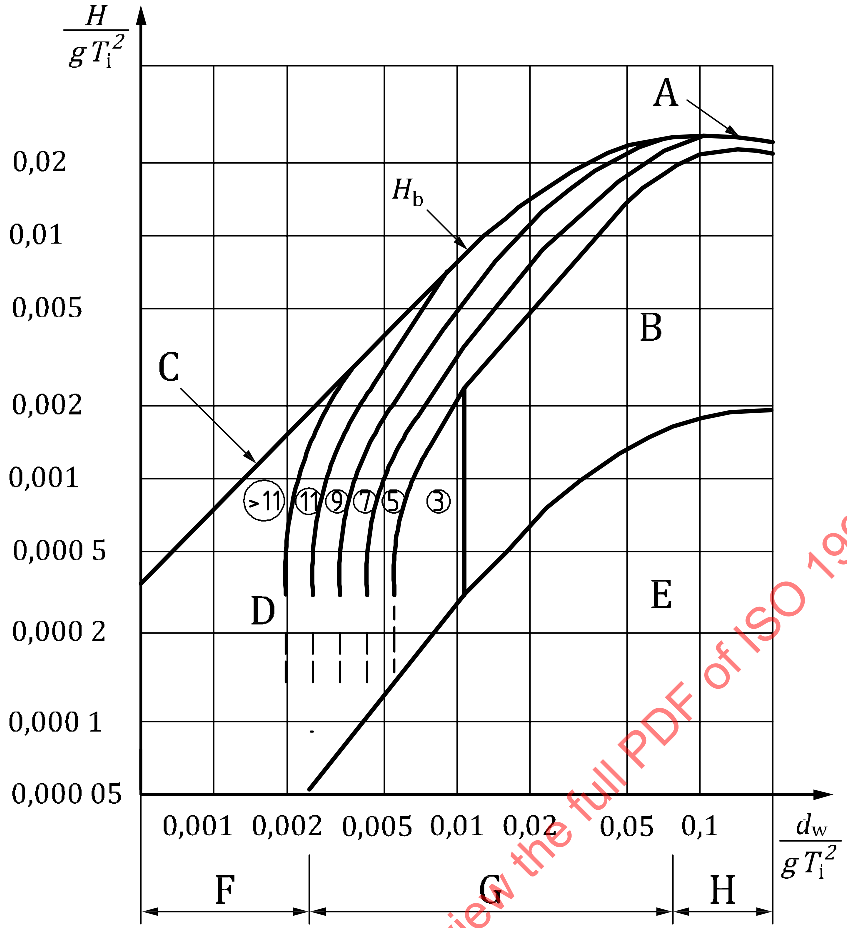
In a dynamic response analysis, the added mass (m_a integrated over the member length) is normally transferred to the left-hand side of the formula of motion and added to the structural mass.

A.7.3.3.3 Wave models

A.7.3.3.3.1 Deterministic waves

For deterministic analyses an appropriate wave theory for the water depth, wave height and period should be used, based on the curves from ISO 19901-1:2015, A.8.4.2, as shown in Figure A.7.3-5. For practical purposes, Stokes' 5th (within its bounds of applicability) or an appropriate order of Dean's Stream Function are acceptable for regular wave elevated storm analysis.

If breaking waves are indicated according to ISO 19901-1:2015, A.8.4.2, it is recommended that the wave period is changed to conform with the breaking limit for the specified height.



Key

d_w water depth

g acceleration due to gravity

H maximum wave height

H_b breaking wave height

T_i intrinsic wave period

A deep water breaking limit <<$H/L_w = 0,14$, where L_w is the wave length

B Stokes' fifth order, New-wave or third order stream function

C shallow water breaking limit $H/d_w = 0,78$

D stream function (showing order number)

E linear/Airy or third order stream function

F shallow water

G intermediate depth

H deep water

NOTE Taken from ISO 19901-1:2015.

Figure A.7.3-5 — Regions of applicability of alternative wave theories

A.7.3.3.3.2 Stochastic waves

Time domain analysis is recommended for stochastic wave jack-up analysis. In such analyses the waves are modelled using a random superposition model to represent the wave spectrum; see A.6.4.2.5 to A.6.4.2.8. It is recommended that the random sea state be generated from the summation of at least 200 component waves of height and frequency determined to match the wave spectrum. The phasing of the component waves should be selected at random. A two-dimensional first order simulation using linear (Airy) waves is normally sufficient. However, when the effects of wave spreading is explicitly included in the analysis method, a three-dimensional simulation using a higher order wave theory should be used to capture higher frequency interaction effects (e.g. those due to frequency sum terms).

For first order wave kinematic models, the extrapolation of the wave kinematics to the free surface (wave stretching) is most appropriately carried out by substituting the true elevation at which the kinematics are required with one which is at the same proportion of the still water depth as the true elevation is of the instantaneous water depth. This can be expressed as given in Formula (A.7.3-18):

$$z' = \frac{z_\mathrm{k} - \zeta_\mathrm{w}}{1 + \frac{\zeta_\mathrm{w}}{d_\mathrm{w}}} \tag{A.7.3-18}$$

where

z' is the modified coordinate for use in particle velocity formulation;

z_k is the vertical coordinate relative to the SWL under consideration, positive upwards, at which the kinematics are required;

ζ_w is the instantaneous water level (same axis system as z);

d_w is the water depth, still or undisturbed (positive).

This method ensures that the kinematics at the instantaneous free surface are always evaluated from the linear wave theory expressions as if they were at the still water level, see Wheeler (1969)[199] and ISO/TR 19905-2:2012, A.7.3.3.3.2.

For higher order wave-kinematic models, an appropriate alternative for stretching the wave profile to the instantaneous wave surface should be adopted.

The statistics of the underlying random wave process are Gaussian and fully known theoretically. The empirical modification around the free surface to account for free surface effects, together with the fact that drag actions are a non-linear (squared) transformation of wave kinematics, makes the hydrodynamic action excitation always non-linear. As a result, the random excitation is non-Gaussian. The statistics of such a process are generally not known theoretically, but the extremes are generally larger than the extremes of a corresponding Gaussian random process. For a detailed investigation of the dynamic behaviour of a jack-up, the non-Gaussian effects should be included. Multiple procedures for doing this are presented in Annex C.

When the random displacements of the submerged parts are small and the velocities are significant with respect to the water-particle velocities, the damping is not well represented by the relative velocity formulation in the Morison equation, which tends to overestimate the damping and underpredict the response. A criterion for determining the applicability of the relative velocity formulation is given in A.7.3.3.2.

A summary of recommendations for the time domain modelling of random waves is given in Table A.7.3-3.

Table A.7.3-3 — Recommendations for modelling of time domain stochastic waves

Method	Recommendations
Time domain	Generate random sea from at least 200 components and use divisions of generally equal energy. It is recommended that smaller energy divisions be used in the higher frequency portion of the spectrum, which generally contains the reinforcement and cancellation frequencies. For each component, the relationship between wave length and frequency should be taken according to its linear dispersion relationship (Sarpkaya, 1981)[160]. Unless indicated otherwise in the site-specific information, the validity of wave surface simulation should be checked against the criteria given below. The criteria for higher order waves should be taken to ensure that H_s, mean waves and maximum crests are within practical limits. — correct mean wave elevation; — standard deviation = $(H_s/4) \pm 1$ %; — $-0{,}03 < \text{skewness} < 0{,}03$; — $2{,}9 < \text{kurtosis} < 3{,}1$; — maximum crest elevation = $(H_s/4)\sqrt{2\ln(N_C)}$ −5 % to +7,5 %; where N_C is the number of cycles in the time series being qualified, $N_C \approx \text{Duration}/T_z$. Integration time step less than the smaller of $T_z/20$ or $T_n/20$ where T_z is the apparent mean zero-upcrossing period of the wave spectrum; T_n is the jack-up natural period, see A.10.4.2.1 (unless it can be shown that a larger time step leads to no significant change in results). Avoid transient effects, discard at least the first 100 s (the "run-in"). Ensure the simulation is of sufficient duration so that the method chosen results in demonstrably stable MPME responses; see also A.10.5.3.4 and C.2.

A.7.3.3.3.3 The effect of directionality and spreading on dynamic response

Both the magnitude of the actions on the structure and the dynamic amplification are affected by cancellation and reinforcement of wave actions, dependent on leg spacing (heading) and wave length. The effects of directionality and wave spreading should therefore be considered in any random dynamic analysis. The following two methods can be used to develop a representative DAF in conjunction with adjustments to the natural period (A.10.4.2.5.3).

Method 1: In a two-dimensional long-crested simulation, the effect of directionality can be included by developing a base shear transfer function (BSTF) accounting for spreading, "BSTF with spreading", as described below [see 7.6.4 of Sarpkaya (1981)[160]].

a) Develop a set of two-dimensional BSTFs, one for the "principal" direction of interest, and the others offset from the principal direction.

b) For each offset direction, calculate a directionality contribution factor from ISO 19901-1:2015, A.8.3.2.1, or from ISO/TR 19905-2:2012, 6.4.2.8. Each factor corresponds to a given percentage of area under the directionality function such that the sum of all the factors is 1,0.

c) The "BSTF with spreading" is then the sum of each two-dimensional BSTF (principal one plus the offset directions) multiplied by the corresponding directionality factors. Be aware that only the principal direction vector component of the offset direction BSTFs is used.

d) The BSTF for the chosen two-dimensional (long-crested/unspread) analysis direction and the "BSTF with spreading" are compared to determine whether the selected direction is unconservative. Optimally, the direction of the two-dimensional sea state should be chosen to obtain a match with the three-dimensional BSTF for the entire wave frequency range. If this is not possible, the match between the spread and unspread BSTFs should be good at the natural period.

Method 2: To minimize reinforcement and cancellation effects, it is suggested that the dynamic analysis be carried out for a single wave heading along an axis that is neither parallel nor normal to a line through two adjacent leg centres. Thus, for a 3-legged jack-up with equilateral leg positions and a single bow leg, suitable analysis headings can be with the weather approaching from approximately 15° or 45° off the bow. The DAFs should be determined for one, or both, of these headings with suitably adjusted natural period; see Figure A.10.4-1. The DAFs (or more conservative DAFs) can then be applied to the final deterministic analysis for all headings.

A.7.3.3.4 Current

The current velocity and profile as specified in A.6.4.3 should be used. Where the current profile is defined by discrete points, linear interpolation between the data points is sufficient.

The current induced drag actions are determined in combination with the wave actions. This is carried out by the vectorial addition of the wave and current induced particle velocities prior to the drag action calculations.

The current velocity may be reduced to account for interference from the structure with the flow field of the current, as given in Formula (A.7.3-19); see Taylor (1991)[179] and ISO/TR 19905-2:2012, 7.3.3.4:

$$V_C = V_f[1 + C_{De}D_e / (4D_F)]^{-1} \quad \text{(A.7.3-19)}$$

where

V_C is the current velocity for use in the hydrodynamic model; V_C should not be taken as less than 0,7V_f;

V_f is the far field (undisturbed) current velocity;

C_{De} is the equivalent drag coefficient of the leg, as defined in A.7.3.2;

D_e is the equivalent diameter of the leg, as defined in A.7.3.2;

D_F is the face width of leg, outside dimensions, orthogonal to the flow direction.

A.7.3.3.5 Intrinsic and apparent wave periods

The intrinsic wave period is based on a reference frame travelling with the speed and direction of the current, and should be used, except as detailed later in this subclause, to calculate the wave kinematics. The apparent wave period is that which is observed by a stationary observer and is the period that should be used to calculate the jack-up dynamics. The intrinsic wave period, in conjunction with the water depth and appropriate wave theory, are used to calculate the wave length.

NOTE 1 There is only the intrinsic wave length; there is no apparent wave length. If one applies the apparent wave period in an analysis, the excitation period is correct but both the kinematics and the wave length are wrong. The wrong wave length means that the legs of a jack-up are at the wrong relative positions in the wave. The conceptual solution is to model the un-modified intrinsic wave with the jack-up moving into the wave at the current velocity.

It is important to determine whether the supplied wave period is apparent or intrinsic, taking due care to ensure that ISO 19901-1 terminology is consistently adhered to at all times. ISO 19901-1 terminology can conflict with the definition of these terms used by the supplier of the metocean data.

NOTE 2 ISO 19901-1 uses terminology conflicting from that in API RP 2A-LRFD, (1993)[14]. In ISO 19901-1, the “apparent” wave period is defined as the wave period seen by a stationary observer, while the “intrinsic” wave period is the wave period seen by an observer moving with the current. In API RP 2A the “actual” wave period is defined as the wave period seen by a stationary observer, while the “apparent” wave period is the wave period as it “appears” to an observer moving with the current. By comparison, ISO 19901-1 “intrinsic” equates to RP 2A “apparent”, and ISO 19901-1 “apparent” equates to RP 2A “actual”.

Formulae for transformation between the intrinsic and apparent wave periods are given in ISO 19901-1:2015, A.8.4.3. It gives no direct guidance on modifying short-crested sea states, although a suitable method can be inferred. The assessor should ensure that the correct procedure is used by the software in calculating wave particle kinematics and dynamics; it is important to understand the terminology used by the software vendor; see NOTE 2. In summary, the steps taken to convert intrinsic to apparent wave period are as follows.

a) Calculate the wave length based on the intrinsic wave period and the water depth, using a suitable wave theory.

b) Calculate the intrinsic wave celerity as wave length divided by intrinsic wave period.

c) Calculate the apparent wave celerity by adding the resolved current velocity to the wave celerity (the celerity is increased if the current is in the same direction as wave propagation, and decreased if in an opposing direction).

d) Calculate the apparent wave period as the wave length divided by the apparent celerity.

Conversion from an apparent wave period to an intrinsic wave period follows a similar approach but is undertaken iteratively.

Care should be taken with opposing currents that the vector sum of apparent celerity and current is always greater than or equal to zero, otherwise the waves move backwards. This is likely to be relevant only for very short period waves when developing the apparent component periods of a random sea state.

This conversion procedure between apparent and intrinsic periods strictly applies in the case of simple uniform currents over the full water depth. It can be used practically if the current is uniform over the top 50 m of the water column. In cases of a non-uniform current profile, a weighted, depth-averaged in-line current speed, $V_{\text{IN-LINE}}$, may be used, as shown in ISO 19901-1:2015, A.8.4.3, and Kirby and Chen (1989)[120] and as given in Formula (A.7.3-20):

$$V_{\text{IN-LINE}} = \frac{2k}{\sinh(2kd_{\text{w}})} \int_{-d}^{0} V_{\text{c}}(z) \cos\big(\theta(z)\big) \cosh[2k(z + d_{\text{w}})]\text{d}z \tag{A.7.3-20}$$

where

k is the wave number = $2\pi/L_{\text{w}}$;

L_{w} is the actual wave length (i.e. deep water wave length corrected for water depth);

d_{w} is the water depth;

$V_{\text{c}}(z)$ is the current velocity at depth z;

z is the vertical coordinate relative to the SWL under consideration, positive upwards;

$\theta(z)$ is the angular direction of the current at depth z relative to the wave propagation direction; $\theta(z) = 0{,}0$ when in line.

In a two-stage analysis the deterministic quasi-static wave/current actions should be determined using the intrinsic period.

The apparent wave period should be used for the SDOF DAF calculation of $K_{DAF,SDOF}$.

For stochastic calculations, the rigorous approach is to develop the particle kinematics for the components using the intrinsic wave period and to develop the wave/current actions by applying the intrinsic kinematics to the jack-up by using component wave phases based on the apparent wave period. This approach should be used for one-stage analysis and for two-stage analysis with a non-linear foundation model for the DAF calculations. This procedure is difficult if the available analytical tools do not have the feature implemented.

When undertaking a two-stage deterministic storm analysis (A.10.5.2) using a DAF developed from a random dynamic analysis (A.10.5.2.2.3) with linearized foundations, it can be acceptable to use a spectrum with an apparent peak period for all stages in the calculation of $K_{DAF,RANDOM}$ and the inertial loadset. The error is expected to be small when the ratio $T_{p,i}/T_p$ is within the range $1 \pm 0{,}08$. If this approach is used, the analysis should also be undertaken without period adjustment and the more onerous DAFs used. When $T_{p,i}/T_p$ is outside this range, a more rigorous approach should be considered.

A.7.3.4 Wind actions

A.7.3.4.1 Wind action

The wind action on each component (divided into blocks of not more than 15 m vertical extent), F_{Wi}, can be computed using Formula (A.7.3-21):

$$F_{Wi} = P_i A_{Wi} \tag{A.7.3-21}$$

where

P_i is the pressure at the centre of block i;

A_{Wi} is the projected area of block i perpendicular to the wind direction.

The pressure P_i should be computed using Formula (A.7.3-22):

$$P_i = 0{,}5 \rho_a V_{Zi}^2 C_s \tag{A.7.3-22}$$

where

ρ_a is the mass density of air (taken as 1,222 4 kg/m^3 unless an alternative value can be justified for the site);

V_{Zi} is the specified wind velocity at the centre of block i; see A.6.4.6.2;

C_s is the shape coefficient, as given in A.7.3.4.2.

Wind actions on legs below the hull should be calculated to either the instantaneous wave surface or to SWL.

NOTE The wind area of the hull and associated structures (excluding derrick and legs) can normally be taken as the projected area viewed from the wind direction under consideration.

A.7.3.4.2 Shape coefficient

Using building block elements, the shape coefficients in Table A.7.3-4 should be used.

Table A.7.3-4 — Shape coefficients

Type of member or structure	Shape coefficient, C_s
Hull side (flat side)	1,0 based on total projected area
Hull and associated structures (excluding derrick and legs)	1,1 based on the total projected area (i.e. the area enclosed by the extreme contours of the structure)
Deckhouses, jack-frame structure, sub-structure, draw-works house, and other above-deck blocks	1,1 based on the projected area
Leg sections projecting above jack-frame structure and below the hull	$C_s = C_{De}$ as determined from A.7.3.2.3, normally using smooth drag coefficients (ignoring marine growth) A_{Wi} determined from D_e and section length
Isolated tubulars (crane pedestals, etc.)	0,5
Isolated structural shapes (angles, channels, box, I-sections)	1,5 based on member projected area
Derricks, crane booms, flare towers (open lattice sections only, not boxed-in sections)	The appropriate shape coefficient for the members concerned applied to 50 % of the total projected profile area of the item (25 % from each of the front and back faces)
Shapes or combinations of shapes that do not readily fall into the above categories should be subject to special consideration.	

A.7.3.4.3 Wind tunnel data

Wind pressures and resulting actions for the hull and associated structures may be determined from wind tunnel tests on a representative model. Care should be exercised when interpreting wind tunnel data for structures mainly comprised of tubular components, such as truss legs.

A.7.4 Functional actions

Provided appropriate procedures exist and it is practical to change the mode of the jack-up from operating to elevated storm mode on receipt of an unfavourable weather forecast, it is necessary to assess only the elevated storm mode. Consideration should be given to actions on the conductors if supported by the jack-up.

The following should be defined:

a) actions due to the maximum and minimum elevated weight. In the absence of other information, the minimum elevated weight can normally be determined assuming 50 % of the variable load permitted by the operating manual;

b) extreme limits of the centre of gravity position (or reactions of the elevated weight on the legs) for the configurations in a) above;

c) substructure and derrick position, hook load, rotary load, setback and conductor tensions for the configurations in a) above;

d) weight, centre of gravity and buoyancy of the legs.

If a minimum elevated weight or a limitation of the centre of gravity position is required to meet the overturning acceptance criteria (see 5.4.4 and 13.8), then the addition of water in lieu of variable load is permitted in the assessment, provided that

— the functional actions do not exceed the operations manual limits,

— procedures, equipment and instructions exist for performing the operation of adding water offshore, and

— the action due to the maximum variable load, including added water, is used for all appropriate assessment checks (preload, stress, etc.).

If a reduction in elevated weight or a limitation of the centre of gravity position is required to meet the foundation acceptance criteria with respect to foundation sliding, see 5.4.4 and 13.9.1, then the variable load used in the assessment can be revised accordingly provided that procedures, equipment and instructions exist for the timely performance of the operation offshore.

A.7.5 Displacement dependent actions

No guidance is offered.

A.7.6 Dynamic effects

No guidance is offered.

A.7.7 Earthquakes

See 10.7 and A.10.7.

A.7.8 Ice actions

See 10.8 and A.10.8.

A.7.9 Other actions

Other actions should be represented as relevant for the site.

For areas where icing is possible during the planned operation, the effect on weight and on the environmental actions should be considered. Relevant data for the region should be applied. For calculating wave, current and wind actions, increases in dimension and changes in shape and surface roughness can be significant.

A.8 Guidance on structural modelling

A.8.1 Applicability

Techniques for modelling the legs, hull, leg-to-hull connection, and leg/spudcan connection are discussed. The leg-to-hull connection model includes the upper and lower guides, jacking pinions, fixation systems, and jackcase/associated bracing. Modelling of the foundation is limited to the structural details in this clause; geotechnical aspects are presented in A.9.

Because of the interaction of the mass and stiffness models, e.g. the effect of mass modelling on hull sag, it is recommended that the assessor be familiar with the whole of this clause before commencing the modelling.

A.8.2 Overall considerations

A.8.2.1 General

No guidance is offered.

A.8.2.2 Modelling philosophy

The structural model should accurately reflect the complex mechanism of the jack-up; for most jack-up configurations this requires the use of an FE computer model. A.8.3 to A.8.5 describe the structural aspects of the model. A.8.6 describes the interaction of the structural model with the foundation. A.8.7 describes modelling the mass and A.8.8 describes the application of the actions.

A.8.2.3 Levels of FE modelling

While it can be desirable to fully model the jack-up when assessing its structural strength, this is rarely necessary for a site-specific assessment. An overly complex model can introduce errors and unnecessarily complicate the assessment. Consequently, assumptions and simplifications, such as equivalent hull, equivalent leg, etc., are often made when building the model(s) used for the assessment. In view of this, one of the various levels of modelling described in a) through d) below can be used. It should be recognized that some of these methods have limitations with respect to the accuracy of assessing the structural adequacy of a jack-up. Table A.8.2-1 outlines the limitations of the various modelling techniques and should be referenced to ensure that the selected model addresses all aspects required for the assessment. When simplified models, such as those described in b) and d) are used, it is usually appropriate to calibrate them against a more detailed model.

a) Fully detailed leg model:

   The model consists of “detailed legs”, hull, leg-to-hull connections and spudcans modelled in accordance with A.8.3.2, A.8.4, A.8.5 and A.8.6, respectively. The results from this model can be used to examine all aspects of a jack-up site-specific assessment, including foundation stability, overturning resistance, leg strength and the adequacy of the jacking system or fixation system.

b) Equivalent leg (stick model):

   The model consists of “stick model” legs (see A.8.3.3), hull structure modelled using beam elements (see A.8.4.3), leg-to-hull connections (see A.8.5) and spudcans modelled as a stiff or rigid extension to the equivalent leg. The results from this model can be used to examine foundation stability and overturning resistance. This model can also be used to obtain reactions at the spudcan and internal forces and moments in the leg in the vicinity of the lower guide for application to the “detailed leg” and leg-to-hull connection model d).

c) Combined equivalent/detailed leg and hull model:

   The model consists of a combination of “detailed leg” for the upper portion of legs and “stick model” for the lower portion of the legs (see A.8.3.4). The hull, leg-to-hull connections and spudcans are modelled in accordance with A.8.4, A.8.5 and A.8.6, respectively. The results from this model can be used to examine foundation stability, overturning resistance, leg strength in the region of the leg-to-hull connections and the adequacy of the jacking and/or fixation systems. See Figure A.8.2-1.

d) Detailed single leg and leg-to-hull connection model:

   The model consists of a “detailed leg” or a portion of a “detailed leg” (see A.8.3.2), the leg-to-hull connection (see A.8.5) and, when required, the spudcan (see A.8.6). The results from this model can be used to examine the leg strength and the adequacy of the jacking and/or fixation systems.

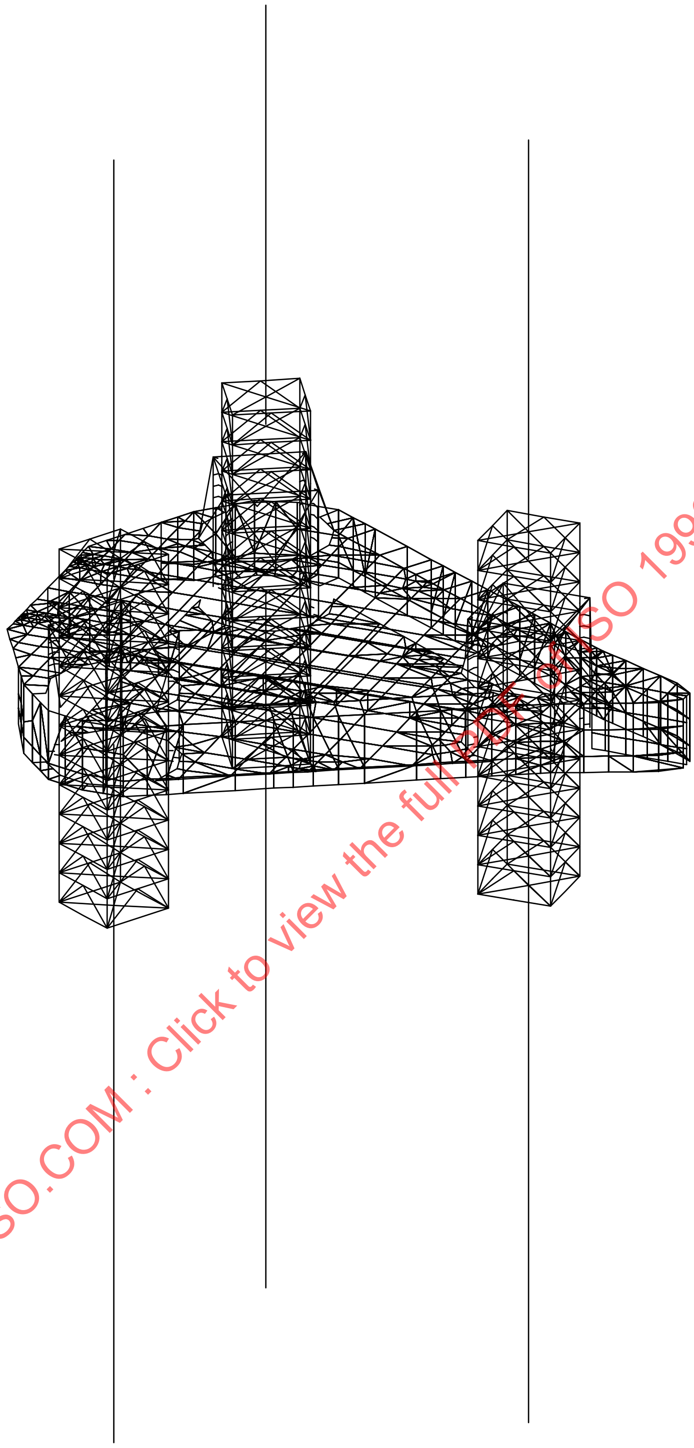


Figure A.8.2-1 — Combined equivalent/detailed leg and hull model

Table A.8.2-1 — Applicability of the suggested models

Model type	Applicability						
	I	II	III	IV	V	VI	VII
	Base shear and overturning moment	Overturning checks	Foundation checks	Global leg forces	Leg member forces	Jacking/fixation system reactions	Hull element forces
a) Fully detailed leg	Yes	Yes	Yes	Yes	Yes	Yes	See note
b) Equivalent leg (stick model)	Yes	Yes	Yes	Yes	—	—	—
c) Combined equivalent/detailed leg and hull	Yes	Yes	Yes	Yes	Yes	Yes	See note
d) Detailed single leg and leg-to-hull connection model	—	—	—	—	Yes	Yes	—
NOTE Hull stresses are only available from more complex hull models.							

A.8.3 Modelling the leg

A.8.3.1 General

For truss legs the model(s) can be generated in accordance with A.8.3.2 to A.8.3.4 as applicable. Single column legs can be modelled with beam elements (see A.8.3.3) or by means of other appropriate finite elements with due consideration for local and global buckling.

A.8.3.2 Detailed leg

Modelling should account for offsets between member work points and centroids, as omitting this detail can be unconservative. If member offsets are not included in the model, analysis of the relevant joints should consider their effect. Gusset plates are typically omitted in the structural leg model. However, their beneficial effects can be taken into account in the calculation of member and joint strength.

A.8.3.3 Equivalent leg (stick model)

The leg structure can be simulated by a series of collinear beams with the equivalent cross-sectional properties calculated using the formulae indicated in Tables A.8.3-1 and A.8.3-2 or derived from the application of suitable unit load cases to the 'detailed leg'. The stiffness properties of the equivalent leg should equate to those of the 'detailed leg' model described in A.8.3.2. Where such a model is used, relevant analysis results can be applied to a detailed leg model to determine member stresses, fixation system/pinion forces, etc.

The determination of stiffness for the equivalent leg model can be accomplished as outlined below.

a) From hand calculations using the formulae presented in Tables A.8.3-1 and A.8.3-2. If the leg scantlings change in different leg sections, this can be accounted for by calculating the properties for each leg section and creating the equivalent leg model accordingly. Provided that there are no significant offsets between the brace work points, these are reasonably accurate for cases A (sideways K bracing), C (X bracing) and D (Z bracing). Case B (normal K bracing) should be used with caution as the values of equivalent shear area and second moment of area are dependent on the number of bays being considered.

b) From the application of unit load cases to a detailed leg model prepared in accordance with 8.3.2 and 8.3.5: The leg should be rigidly restrained, generally at the first point of lateral force transfer between the hull and the leg, although it can be more convenient to use a different reference point, e.g. level of the fixation system or neutral axis of the hull. The variables Δ, δ_M, θ_M and θ_P used in Formulae (A.8.3-1) to (A.8.3-4) are obtained from the detailed leg model. The following load cases should be considered, applied about the major and minor axes of the leg:

— Axial unit load case: This is used to determine the axial area, A_{eq}, of the equivalent leg model beam according to standard beam theory as given in Formula (A.8.3-1):

$$\Delta = \frac{FL_c}{A_{eq}E} \Rightarrow A_{eq} = \frac{FL_c}{E\Delta} \tag{A.8.3-1}$$

where

Δ is the axial deflection (shortening) of the cantilever at the point of force application;

F is the applied axial action;

L_c is the cantilevered length from the hull to the seabed reaction point; see A.8.6.2;

E is Young's modulus of steel.

— Pure moment applied either as a moment or as a couple at the end of the cantilever: This is used to derive the second moment of area (I) according to standard beam theory as given in Formula (A.8.3-2):

$$\delta_M = \frac{ML_c^2}{2EI} \Rightarrow I = \frac{ML_c^2}{2E\delta_M} \quad \text{and} \quad \theta_M = \frac{ML_c}{EI} \Rightarrow I = \frac{ML_c}{E\theta_M} \tag{A.8.3-2}$$

where

δ_M is the lateral deflection of the cantilever at the point of moment application;

M is the applied moment;

θ_M is the slope of the cantilever at the point of moment application.

It should be recognized that the value of I resulting from the two formulae can differ somewhat.

— Pure shear, P, applied at the end of the cantilever, which can be used to derive I according to standard beam theory as given in Formula (A.8.3-3):

$$\theta_P = \frac{PL_c^2}{2EI} \Rightarrow I = \frac{PL_c^2}{2E\theta_P} \tag{A.8.3-3}$$

where

P is the applied shear;

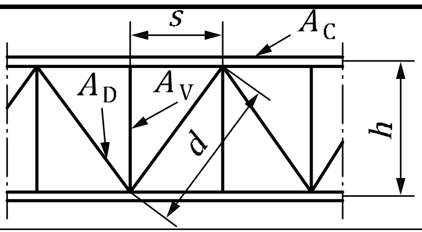
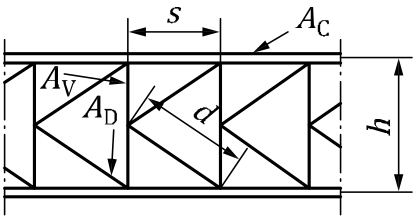
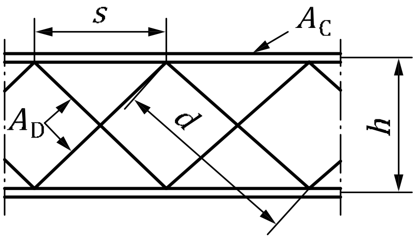
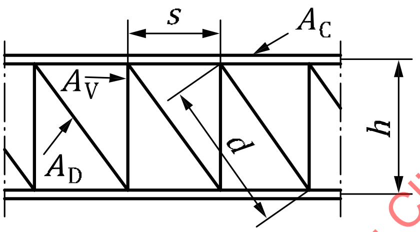
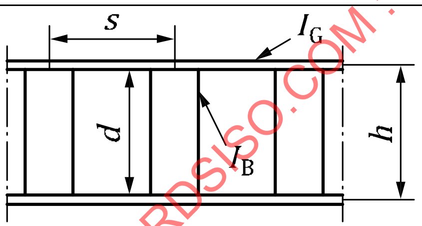
θ_P is the slope of the cantilever at the point of shear application.

Using either this value of I, or a value obtained from the pure moment case, the effective shear area, A_{seff}, of the equivalent leg model beam can then be determined using Formula (A.8.3-4):

$$\delta_M = \frac{PL_c^3}{3EI} + \frac{PL_c}{A_{seff}G} \Rightarrow A_{seff} = \frac{7{,}8PL_cI}{3EI\delta_M - PL_c^3} \tag{A.8.3-4}$$

where G is the shear modulus of steel, $G = E/2{,}6$ for Poisson's ratio of 0,3 for steel.

Table A.8.3-1 — Formulae for determining the effective shear area for two dimensional structures

	Structure	Effective shear area of frame, A_{si}
A		$A_{si}=\dfrac{(1+\nu)sh^2}{\dfrac{d^3}{2A_D}+\dfrac{s^3}{6A_c}}$
B		$A_{si}=\dfrac{(1+\nu)sh^2}{\dfrac{d^3}{A_D}+\dfrac{h^3}{8A_v}-\dfrac{s^3}{NA_c}\left(\dfrac{N^3}{3}-\sum_{i=1}^{N}i^2\right)}$
C		$A_{si}=\dfrac{(1+\nu)sh^2}{\dfrac{d^3}{4A_D}-\dfrac{s^3}{12A_c}}$
D		$A_{si}=\dfrac{(1+\nu)sh^2}{\dfrac{d^3}{2A_D}+\dfrac{h^3}{2A_v}+\dfrac{s^3}{6A_c}}$
E		$A_{si}=\dfrac{48(1+\nu)I_G}{s^2\left(1+\dfrac{2d^2}{sh}\dfrac{I_G}{I_B}\right)}$

Key

s bay height

h centre to centre of chords on face

d length of diagonal brace on face

A_C area of chord

A_D area of diagonal brace

A_V area of horizontal brace

ν Poisson's ratio (0,3 for steel)

I_G largest inertia of chord

I_B largest inertia of brace

N number of active bays

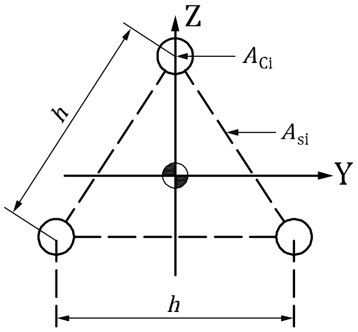
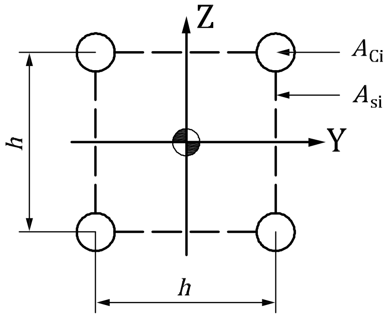
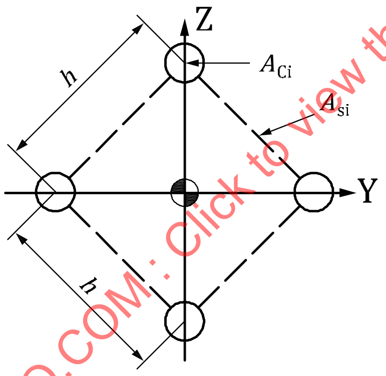
NOTE 1 The stiffness properties are the same for all directions unless the chords have different areas.

NOTE 2 The formulae can be inaccurate if significant offsets exist between brace work points.

NOTE 3 The equivalent beam end rotations can be inaccurate for bracing type C. This can be important if this modelling is used in conjunction with rotational foundation stiffness.

NOTE 4 Based on DNV-RP-C104, (DNV 2022b).

Table A.8.3-2 — Formulae for determining the equivalent section properties of three-dimensional lattice legs

Leg type		Equivalent properties
A		$A_{eq} = 3A_{Ci}$ $A_{sy} = A_{sz} = \frac{3}{2}A_{si}$ $I_y = I_z = \frac{1}{2}A_{Ci}h^2$ $I_T = \frac{1}{4}A_{si}h^2$
B		$A_{eq} = 4A_{Ci}$ $A_{sy} = A_{sz} = 2A_{si}$ $I_y = I_z = A_{Ci}h^2$ $I_T = A_{si}h^2$
C		$A_{eq} = 4A_{Ci}$ $A_{sy} = A_{sz} = 2A_{si}$ $I_y = I_z = A_{Ci}h^2$ $I_T = A_{si}h^2$

Key

A_{si} effective shear area for two-dimensional structure (from Table A.8.3-1)

A_{Ci} individual chord area

A_s effective shear area about representative axis (y or z)

I second moment of area about representative axis (y or z)

I_T torsional moment of inertia

NOTE 1 A_{Ci} can be taken as the cord area including a contribution from the rack teeth (see 8.3.5).

NOTE 2 Based on DNV-RP-C104 (DNV 2022b).

A.8.3.4 Combination of detailed and equivalent leg

The combined detailed and equivalent leg model should be constructed with the areas of interest modelled in detail and the remainder of the leg modelled as an equivalent leg. To facilitate obtaining detailed stresses in the vicinity of the leg-to-hull connection (guides, fixation/jacking system, etc.), the detailed portion of the leg model should extend far enough above and below this region to ensure that boundary conditions at the 'detailed leg'/'equivalent leg' connection do not affect stresses in the areas of interest. Care should be taken to ensure an appropriate interface and consistency of boundary conditions at the connections.

The plane of connection between the "detailed leg" and the "equivalent leg" should remain a plane and without shear distortion when the leg is bent. The connection should be composed of rigid elements that control local bending and shear distortion.

A.8.3.5 Stiffness adjustment

No guidance is offered.

A.8.3.6 Leg inclination

No guidance is offered.

A.8.4 Modelling the hull

A.8.4.1 General

Recommended methods of modelling the hull structure are given in A.8.4.2 and A.8.4.3. Hull mass modelling is discussed in A.8.7 and the modelling of hull sagging is discussed in A.8.8.3.

A.8.4.2 Detailed hull model

The model should be generated using plate elements in which appropriate directional modelling of the effect of the stiffeners on the plates should be included. The elements should be capable of carrying in-plane shear and out-of-plane moment.

A.8.4.3 Equivalent hull model

In an equivalent hull model, the deck, bottom, side shell and major bulkheads are modelled as a grillage of beams. The axial and out-of-plane properties of the beams should be calculated based on the depth of the bulkheads, side shell and the "effective width" of the deck and bottom plating. Beam elements should be positioned with their neutral axes at mid-depth of the hull. Due to the continuity of the deck and bottom structures and the dimensions of a typical hull box, the in-plane bending stiffness can be treated as large relative to the out-of-plane stiffness. The torsional stiffness should be approximated from the closed box section of the hull and distributed between the grillage members.

A.8.5 Modelling the leg-to-hull connection

A.8.5.1 General

The leg-to-hull connection modelling is of extreme importance to the analysis since it controls the distribution of leg bending moments and shears carried between the upper and lower guide structures and the jacking or fixation system. It is, therefore, necessary that these systems be properly modelled in terms of stiffness, orientation and clearance. A simplified derivation of the equivalent leg-to-hull connection stiffness can be used for the equivalent leg (stick model).

A specific jack-up design concept can be described by a combination of the following components (see also Figure C.1-1):

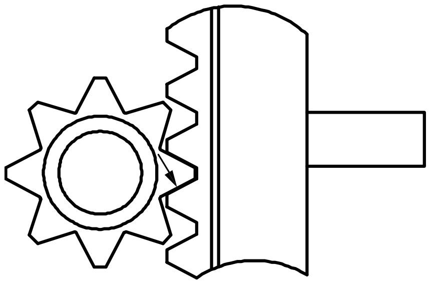
a) with or without fixation system;

b) with opposed jacking pinions [see Figure A.8.5-1 a)];

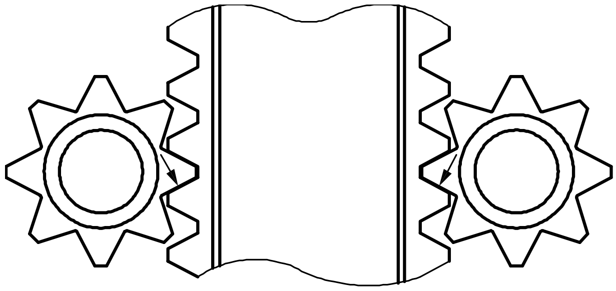
c) with unopposed jacking pinions [see Figure A.8.5-1 b)];

d) with pin and yoke jacking system [see Figure A.8.5-1 c)];

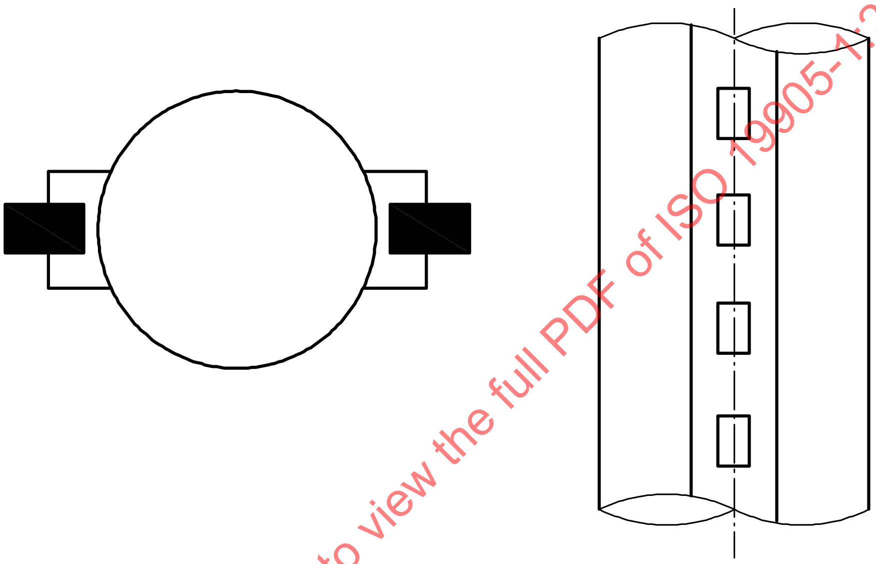
e) with fixed or floating jacking system.



a) Single sided rack and pinion



b) Opposed rack and pinion



c) Pin and yoke

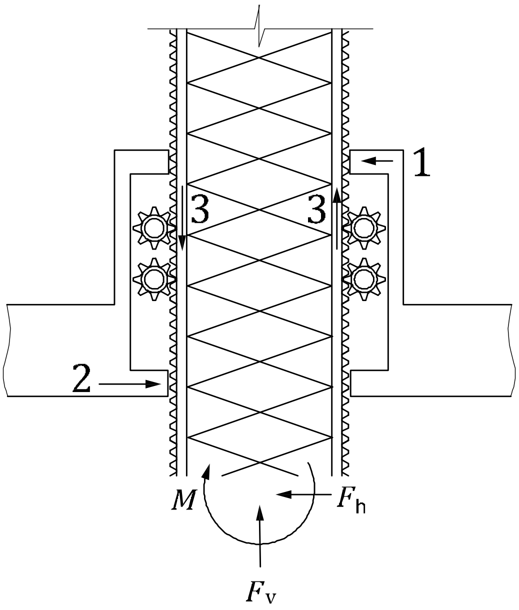
Figure A.8.5-1 — Types of elevating system

Representative leg-to-hull connections are shown in Figure A.8.5-2. The basic function of the leg-to-hull connection is to transfer forces between the leg and hull as follows.

— Horizontal shear is transferred by a set of horizontal forces in the lower guides and/or fixation system.

— Vertical force is transferred via a set of vertical forces in the support system.

— Bending moment is transferred by a combination of horizontal forces in the upper and lower guides and/or by a set of vertical forces in the support system.

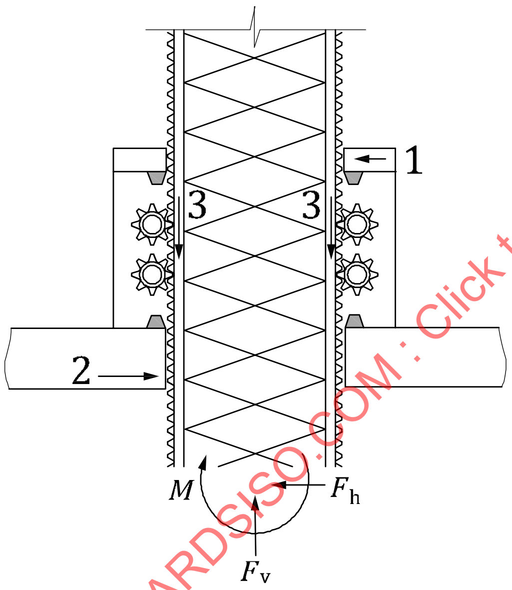


System includes:

— jackcase;

— fixed jacking system with opposed or unopposed jacking pinions.

a) Fixed jacking system without fixation system



System includes:

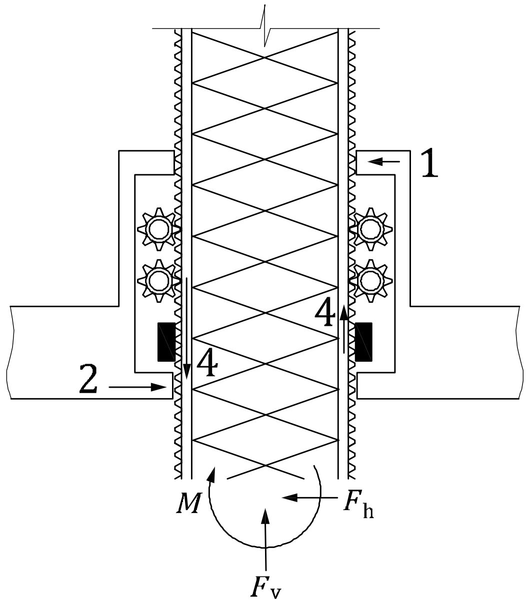
— jackcase;

— shock pads;

— floating jacking system with opposed or unopposed jacking pinions.

b) Floating jacking system without fixation system

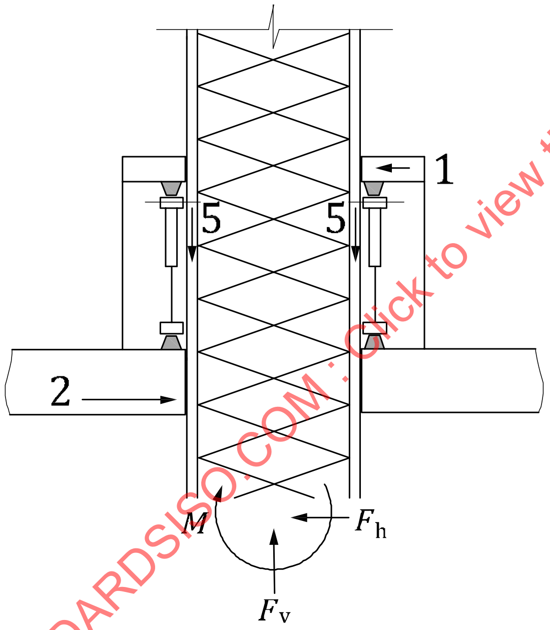
Figure A.8.5-2 — Representative leg-to-hull connections (1 of 2)



System includes:

- jackcase;
- jacking system with opposed or unopposed jacking pinions;
- fixation system.

c) Jacking system with fixation system



System includes:

- jackhouse;
- upper and lower yokes;
- upper and lower shock pads;
- jacking cylinders;
- jacking pins.

d) Pin and yoke jacking system

Key

1 upper guide reaction
2 lower guide reaction
3 pinion reactions
4 fixation system reactions
5 jacking pin reactions
F_v axial force in leg at lower guide
F_h shear force in leg at lower guide
M bending moment in leg at lower guide

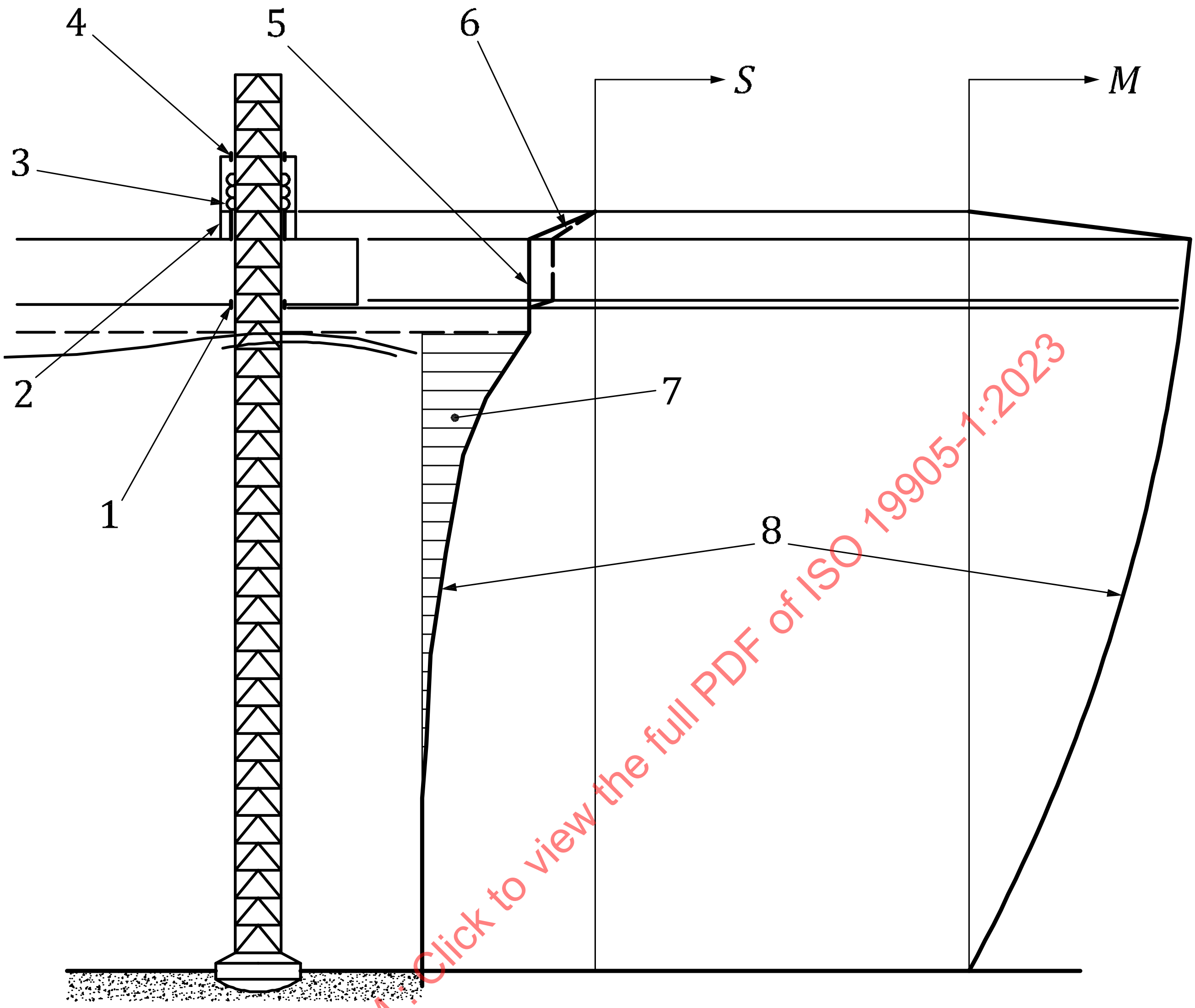
Figure A.8.5-2 — Representative leg-to-hull connections (2 of 2)

For jack-ups with a fixation system, the leg bending moment is shared by the upper and lower guides, the jacking system and the fixation systems. Normally, the leg bending moment and the axial force at the leg-to-hull connection due to the environmental actions are transferred largely by the fixation system because of its high stiffness. Depending on the specified method of operation, the stiffnesses, the initial clearances and the magnitude of the applied forces, a portion of the environmental leg loading can also be transferred by the jacking system and the guide structures. After the fixation systems are engaged, some jack-ups release the pinions by disengaging the jacking system. Under this condition, the leg bending moment is shared by the upper and lower guides and the fixation systems. A complete typical shear force and bending moment diagram is shown in Figure A.8.5-3, with a more detailed representation shown in Figure A.8.5-4. In Figure A.8.5-4 a) to c) the part below the lower guide is independent of the leg-to-hull connection.

For jack-ups without a fixation system, the leg bending moment is shared by the jacking system and guide structure. For jack-ups with a fixed jacking system, the distribution of leg moment between the jacking system and guide structure mainly depends on the stiffness of the jacking pinions. Typical shear force and bending moment diagrams for this configuration are shown in Figures A.8.5-4 b) and A.8.5-4 c).

For a floating jacking system, the distribution of leg bending moment between the jacking system and guide structure depends on the combined stiffness of the shock pads and pinions. Typical shear force and bending moment diagrams for this configuration are shown in Figure A.8.5-4 d).

The leg-to-hull connection should be modelled considering the effects of guide and support system clearances, wear, construction tolerances and backlash (within the gear train and between the drive pinion and the rack).



Key

1 lower guide

2 fixation system lower

3 jacking pinion

4 upper guide

5 shear force without lower guide contact

6 shear force with lower guide contact

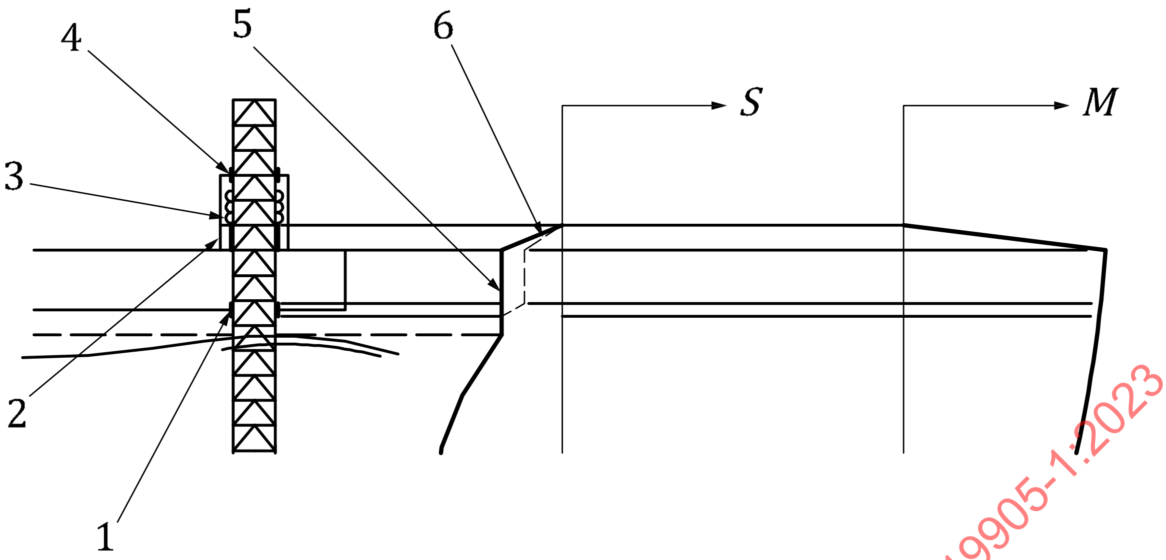
7 shear due to wave/current action

8 net shear or bending moment

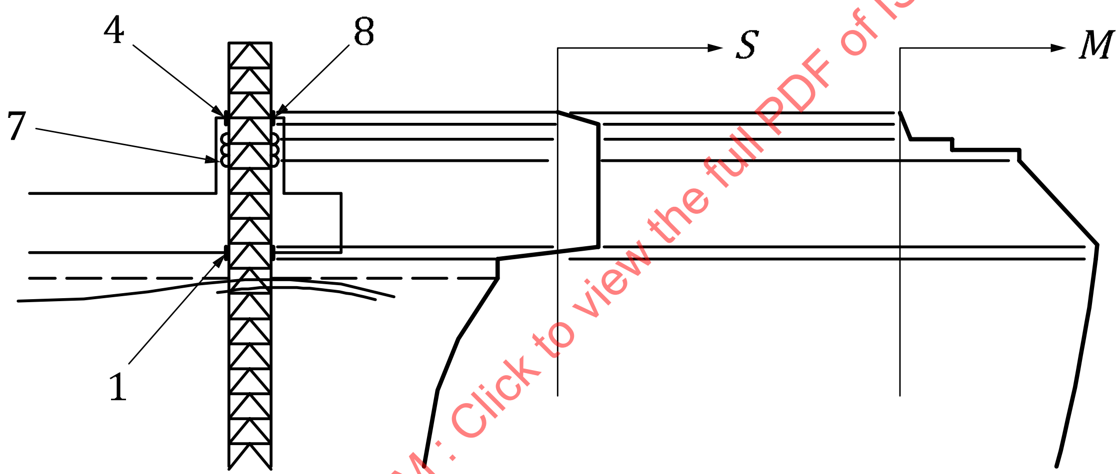
S shear force

M bending moment

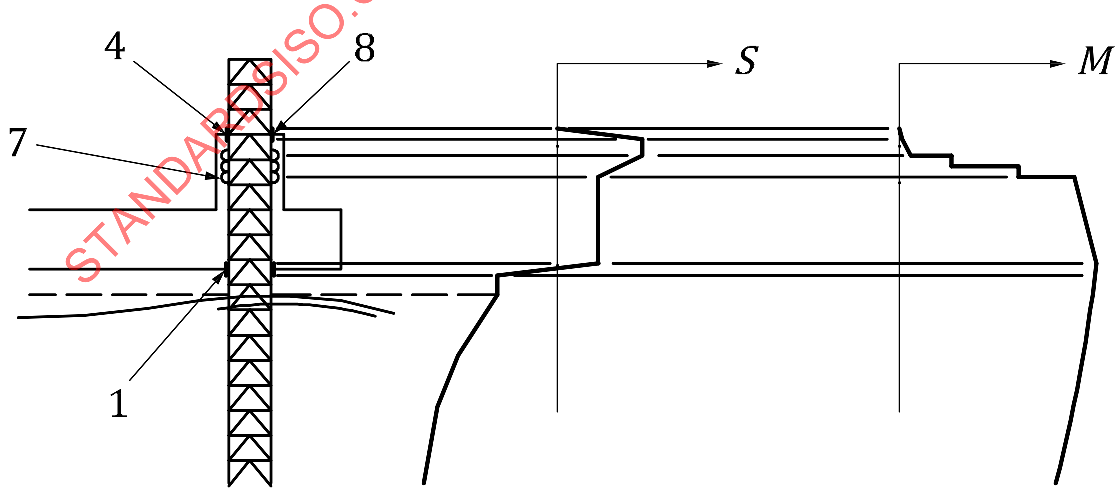
Figure A.8.5-3 — Complete leg shear force and bending moment — Jack-ups with a fixation system



a) Jack-ups with a fixation system

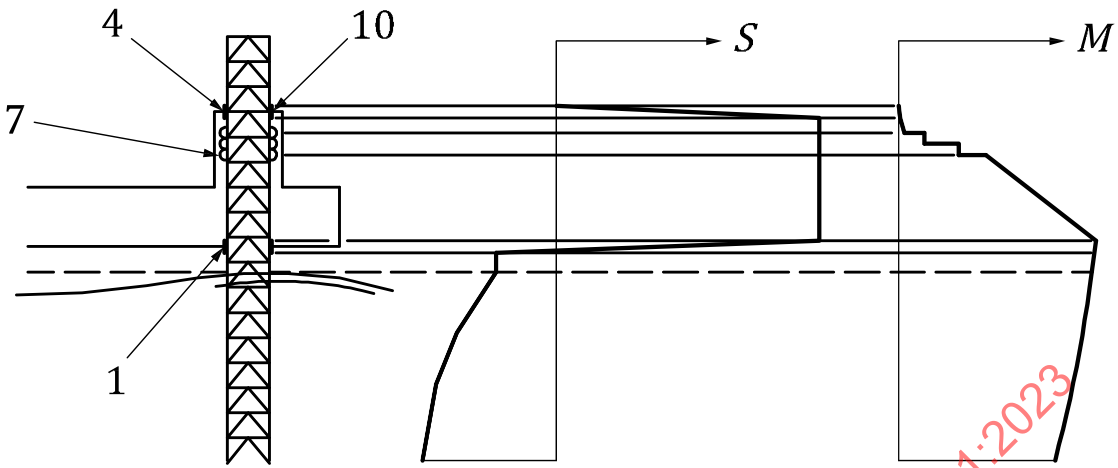


b) Jack-ups without a fixation system and having a fixed jacking system with opposed pinions



c) Jack-ups without a fixation system and having a fixed jacking system with unopposed pinions

Figure A.8.5-4 — Leg shear force and bending moment within the leg-to-hull connection (1 of 2)



d) Jack-ups without a fixation system and having a floating jacking system

Key

1	lower guide	6	shear force with lower guide contact
2	fixation system lower	7	opposed pinions
3	jacking pinion	8	jack case rigidly fixed to hull
4	upper guide	9	unopposed pinions
5	shear force without lower guide contact	10	jack case floating on shock pads

S shear force

M bending moment

Figure A.8.5-4 — Leg shear force and bending moment within the leg-to-hull connection (2 of 2)

If the jacking system has unopposed pinions, local chord moments arise due to

— the horizontal pinion force component (due to the pressure angle of the rack/pinion), and

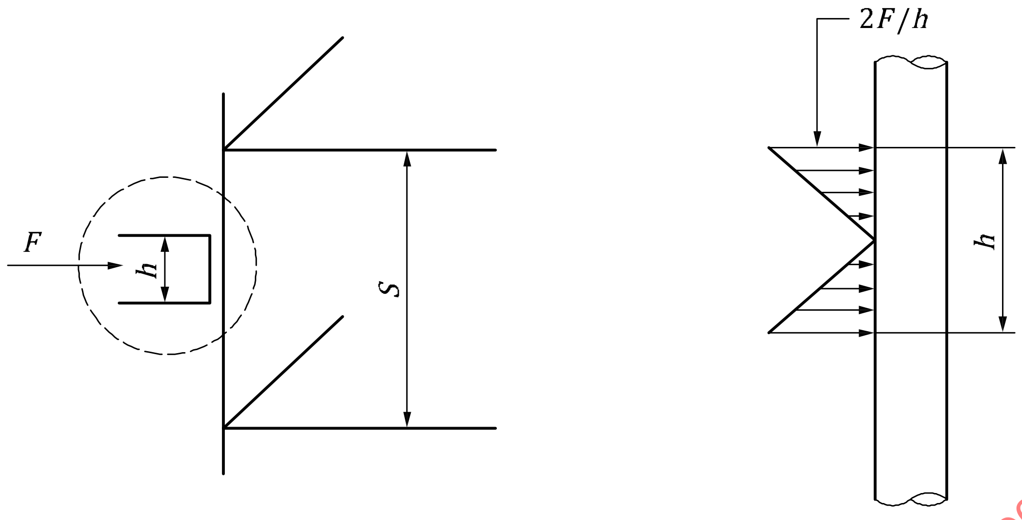
— the vertical pinion force component acting at an offset from the chord neutral axis.

The techniques in A.8.5.2 to A.8.5.7 are recommended for modelling leg-to-hull connections (specific data for the various parts of the structure can be available from the design data package).

A.8.5.2 Guide systems

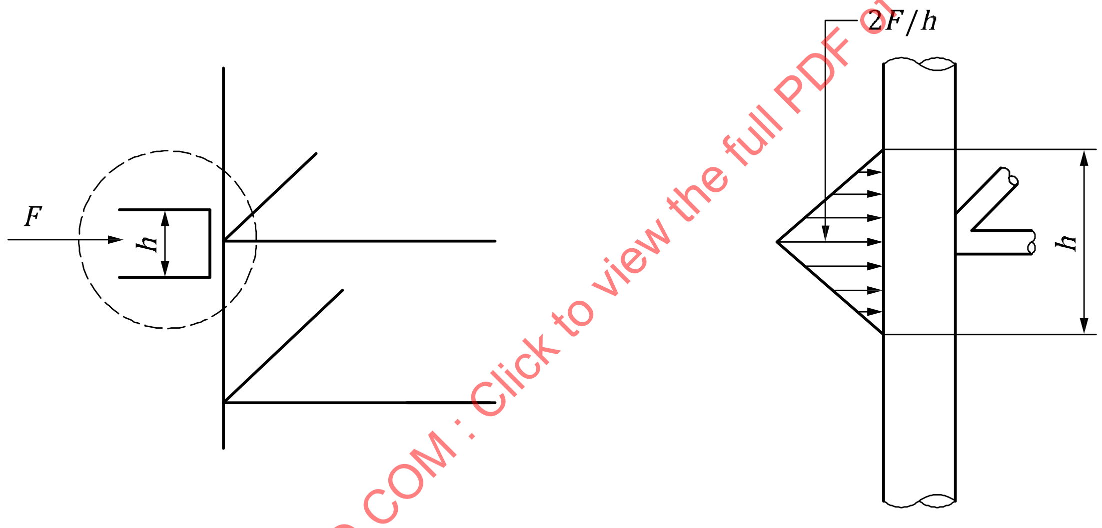
The guide structures should be modelled to restrain the chord member horizontally only in directions in which guide contact occurs. The upper and lower guides can be considered to be relatively stiff with respect to the adjacent structure, such as jackcase, etc. The nominal lower guide position relative to the leg can be derived using the sum of leg penetration, water depth and hull elevation. It is, however, recommended that at least two positions be covered when assessing leg strength: one at a node and the other at midspan. This is to allow for uncertainties in the prediction of leg penetration and possible differences in penetration between the legs.

The finite lengths of the guides can be included in the modelling by means of a number of discrete restraint springs/connections to the hull. Care should be taken to ensure that such restraints carry reactions only in directions/senses in which they can act. Alternatively, the results from analyses ignoring the guide length can be corrected, if necessary, by modification of the local bending moment diagram to allow for the proper distribution of guide reaction; see Figure A.8.5-5. The bending moments in the chord members at the guides determined from a finite element analysis ignoring the guide length, as in Figure A.8.5-5 a) and b), can be corrected using beam analysis for the simplified guide reactions, as shown in Figure A.8.5-5 c) and d) respectively.



a) Guide mid-bay-- guide reaction ignoring guide length

c) Guide mid-bay-- simplified distribution of the guide reaction



b) Guide at node-- guide reaction ignoring guide length

d) Guide at node-- simplified distribution of the guide reaction

Key

F guide reaction

h guide length

s bay height

Figure A.8.5-5 — Correction of point supported guide model for finite guide length

A.8.5.3 Elevating system

A.8.5.3.1 Jacking (or elevating) pinions

The jacking pinions should be modelled using the manufacturer specified pinion stiffness, and should be modelled so that the pinions can resist vertical and the corresponding horizontal forces. A linear spring or cantilever beam can be used to simulate the jacking pinion. The force required to deflect the free end of the cantilever beam a unit distance should be equal to the jacking pinion stiffness. The offset of the pinion/rack contact point from the chord neutral axis should be incorporated in the model.

A.8.5.3.2 Other elevating systems

Elevating system designs not included above should be modelled using stiffness values obtained from the manufacturer/designer, by appropriate system testing or by rational analysis with due consideration of member interface gap spacing and mechanical component stiffness.

A.8.5.4 Fixation system

The fixation system should be modelled to resist both vertical and horizontal forces based on the stiffness of the vertical and horizontal supports and on the relative location of their associated foundations. It is important that the model reflects the local moment strength of the fixation system arising from its finite size and the number and location of the supports.

A.8.5.5 Shock pad — Floating jacking systems

Floating jacking systems generally have two sets of shock pads at each jackcase, one located at the top and the other at the bottom of the jackhouse. Alternatively, shock pads can be provided for each pinion or block of pinions. The jacking system is free to move up or down until it contacts the upper or lower shock pad. In the elevated configuration, the jacking system is in contact with the upper shock pad and in the transit configuration it is in contact with the lower shock pad. The stiffness of the shock pad should be based on the manufacturer's data and the shock pad should be modelled to resist vertical force only. It should also be recognized that the shock pad stiffness characteristics are normally non-linear and can change significantly over time.

A.8.5.6 Jackcase and associated bracing

The stiffness of the jackcase and associated bracing should be modelled accurately since it can have a direct impact on the distribution of horizontal forces between the guides and the jacking system. If the hull is not modelled, it is normally sufficient to restrain the base of the jackcase and associated bracing, as well as the foundations of the fixation system and the lower guide structures at their connections to the hull.

A.8.5.7 Equivalent leg-to-hull stiffness

The determination of stiffnesses for the equivalent leg-to-hull connection model can be accomplished by the following means.

- The application of unit load cases to a detailed leg model in combination with a detailed leg-to-hull connection model in accordance with 8.3.2 and 8.5: Unit load cases are applied as described in A.8.3.3. The effective stiffness of the connection can be determined from the differences between the results from the detailed leg model alone (see A.8.3.3) and those from the detailed leg plus leg-to-hull connection model as follows.
  - Axial unit load case: This case is used to determine the vertical leg-to-hull connection stiffness, K_{vh} from the axial end displacements of the detailed leg model, Δ, and the axial end displacements of the combined leg and leg-to-hull connection model, Δ_C, under the action of the same unit load case, F, as given in Formula (A.8.5-1):

$$K_{vh} = \frac{F}{\Delta_C - \Delta} \tag{A.8.5-1}$$

  - Pure moment applied either as a moment or as a couple: This case is used to derive the rotational leg-to-hull connection stiffness, K_{rh} from either the end slopes, Θ_M and Θ_C, or the end deflections, δ_M and δ_C, of the two models under the action of the same end moment, M, as given in Formula (A.8.5-2):

$$K_{rh} = M/(\theta_C - \theta_M) \quad \text{or} \quad K_{rh} = ML/(\delta_C - \delta_M) \tag{A.8.5-2}$$

— Pure shear, which can be used to determine the horizontal leg-to-hull connection stiffness, K_{hh}, in a similar manner, accounting for the rotational stiffness already derived: Normally, the horizontal leg-to-hull connection stiffness can be assumed infinite.

If the model contains non-linearities, e.g. due to the inclusion of gap elements, care should be taken to ensure that suitable magnitudes of unit load cases are applied to accurately linearize the connection stiffness for the final anticipated displacement including wind actions, etc.

A.8.6 Modelling the spudcan and foundation

A.8.6.1 Spudcan structure

When modelling the spudcan, rigid beam elements are considered sufficient to achieve an accurate transfer of the seabed reaction into the leg chords and bracing. It should be noted that, due to the sudden change in stiffness, these rigid beams can cause artificially high stresses at the leg to spudcan connections. Hence, the modelling and selection of element type should be carefully considered when an accurate calculation of leg member stresses is required in this area.

For a strength analysis of the spudcan and its connections to the leg, a detailed model with appropriate boundary conditions should be developed. This analysis can be performed on an independent model of the spudcan.

A.8.6.2 Seabed reaction point

Unless geotechnical analyses demonstrate otherwise, the vertical position of the reaction point at each spudcan should be located at a distance above the spudcan tip equivalent to

a) half the maximum predicted penetration (when spudcan is partially penetrated), or

b) half the height of the spudcan (when the spudcan is fully penetrated).

The legs of an independent leg jack-up can be either assumed to be pinned or supported with translational and rotational foundation springs at the reaction point. The assumed boundary conditions should be clearly stated together with the assumptions for any moment fixity provided to the spudcans by the soil.

The spudcan geometry, sloping seabeds, bottom obstructions, existing spudcan footprints, etc., can result in horizontal eccentricity of the spudcan support. In such cases, the horizontal position (eccentricity) of the reaction point used in the analysis should be established through calculations that consider the spudcan geometry and seabed topology under the action of preload and should, normally, only be taken into account where this is detrimental to the assessment results. In such cases, the strength of the spudcan should also be considered.

Non-symmetrical geometries should be specially considered.

Further discussion on seabed reaction is contained in Clause 9.

A.8.6.3 Foundation modelling

Methods of establishing the degree of rotational restraint, or fixity, at the spudcans are discussed further in Clause 9 and A.9. Upper or lower values should be considered as appropriate for the areas of the structure under consideration.

When it is necessary to check the spudcans, the leg-to-can connection and the lower parts of the leg, appropriate calculations should be carried out to determine the upper bound spudcan moment considering soil-structure interaction. These areas can be checked by assuming that a percentage of the maximum storm leg moment at the lower guide (derived assuming a pinned spudcan) is applied to the spudcan together with the associated horizontal and vertical seabed reaction forces. This percentage should conservatively be taken as not less than 50 %. For such simplified checks, the spudcan-soil interaction can be modelled assuming that the soil is linear-elastic and incapable of taking tensile stress.

For earthquake screening analyses, see A.10.7

A.8.7 Mass modelling

The vertical distribution of mass is important for all dynamic analyses as it affects the lateral inertial actions. Care should be taken when modelling the hull mass to ensure that the horizontal distribution of mass is correct as it affects the yaw response. This is important particularly in fatigue and earthquake analysis. The cantilever position should be considered when distributing the mass.

For earthquake assessments, see A.10.7.

Normally, the correct functional actions cannot be simply obtained from a mass model of the hull and legs with the application of gravity since it is not possible to consistently account for buoyancy, marine growth, added mass, entrapped water, etc. If the mass model is used to develop the functional actions and dynamic response, then extreme care should be taken to ensure that the proper corrections are made to the functional actions. See A.8.8.2 and A.8.8.3.

A.8.8 Application of actions

A.8.8.1 Assessment actions

The assessment follows a partial factor format. The partial action factors are applied to the actions defined in other clauses (i.e. they are action factors, not action-effect factors). The jack-up response is non-linear and, hence, the application of the combined factored actions does not in general develop the same result as the factored combination of individual action effects.

The actions and action effects are discussed in turn below.

A.8.8.2 Functional actions due to fixed load and variable load

The actions on the hull due to fixed load and variable load should be applied to the model in such a manner as to represent their correct vertical and horizontal distribution. The hull functional actions are the hull masses multiplied by the vertical gravitational acceleration. The hull mass distribution can be represented by a combination of self-generated mass and applied point masses at the node points of the model. When redistribution of the hull weight is used to correct for hull sag moment (see A.8.8.3), the correct horizontal weight distribution can be compromised; when this is undesirable, one of the alternative approaches in A.8.8.3 should be used.

The mass and weight modelling of the legs is more complex than for the hull (see A.8.7). Separate mass and functional action models should consistently account for buoyancy, marine growth, added mass, entrapped water, etc.

In benign areas, the ULS environment is sometimes within the defined SLS limits for the jack-up and the assessment metocean conditions do not exceed the limits for changing to the elevated storm mode (see 5.3). In such cases, the assessment should be for the ULS environment and the proposed operating mode configurations, e.g. with increased variable load, cantilever extended and unequal leg loads. Individual leg reactions under the functional actions can approach the preload reaction. A small additional leg reaction due to environmental actions can then result in additional spudcan penetration.

When the operations manual permits the variable load to be increased as metocean conditions reduce, the jack-up should be assessed to the ULS for operational environments and/or lower return periods (see 5.3). This is of particular importance in areas where significant additional penetrations are possible.

A.8.8.3 Hull sagging

When a jack-up is installed on site, the legs normally engage the seabed with the hull supported by its own buoyancy in a hogged condition. Subsequently, with the hull slightly clear of the water, preload ballast is taken on board thus preloading the legs to achieve their final penetration. This normally leads to an extreme hull sagging condition. Finally, the preload ballast is dumped and the hull elevated to the required elevation for the site. In this configuration, the hull is sagging under self-weight and variable load. The leg shear and bending moments caused by hull sagging are very dependent on leg guide clearances, the design and operation of the jacking system operational parameters, etc. Such moments should be considered in the assessment analyses, and are larger in shallow waters where the leg extension below the hull is small and consequently the leg bending stiffness is higher.

An FE model with distributed hull stiffness and distributed functional actions incorporates hull sag effects if the functional actions are applied to the jack-up in its initially undeflected shape at the operating hull elevation. The hull sag moment is generally overpredicted by this modelling technique and may be reduced by up to 75 % of the value that would be obtained from an analysis using a hull model with

a) the maximum extreme storm weight distributed according to A.8.8.2,

b) guide clearances set to zero, and

c) the elevating system loads equalized within each leg.

The reduction of the hull sag moment should be achieved by one or more of the following:

— applying correcting moments to the hull in the vicinity of each leg;

— redistributing the hull weight, whilst maintaining the correct centre of gravity;

— including realistic guide clearances; and/or

— adjusting position of the spudcan reaction point (prescribed displacement).

Methods that affect the stiffness of the model such as increasing the hull stiffness or increasing the conformity at the base of the legs should be avoided.

If the jack-up is to be operated in an area where the assessment storm falls within its operating limits (as opposed to between operating and survival limits, see 5.3), and for all earthquake assessments, the hull sag moment should be based on the operating condition. This is found as above with the addition of the full effects due to the increase in hull weight and the revised distribution, e.g. 25 % of the initial hull sag plus 100 % of the sag due to the change to the operating condition.

A.8.8.4 Metocean actions

A.8.8.4.1 Wind actions

Wind actions are determined from 7.3.4. The wind actions on the legs above and below the hull should be modelled to represent their correct vertical and horizontal distribution. Actions can be applied as distributed or as nodal actions. Where nodal actions are used, a sufficient number should be applied to reflect the distributed nature of the actions, and it should be ensured that the correct total shear and overturning moment are achieved on each leg.

Similarly, the wind actions on the hull and associated structure can be applied as distributed or as nodal actions. The application should also ensure that the correct total shear and overturning moment on the hull are achieved.

A.8.8.4.2 Wave/current actions

Wave/current actions are determined from 7.3.3. The wave/current actions on the leg and the spudcan structures above the sea floor should be modelled to represent their correct vertical and horizontal distribution. Where nodal actions are used, their application should ensure that the correct total shear and overturning moment are achieved on each leg, and reflect the distributed nature of the actions.

A.8.8.5 Inertial actions

A deterministic dynamic storm analysis requires the explicit determination of an inertial loadset, F_{in} (see Clause 10). This loadset should be applied to the model in combination with the other actions.

For the SDOF approach, F_{in} is applied to the hull as lateral force(s) acting through the hull centre of gravity.

When the inertial loadset is derived from a random dynamic analysis, the applied loadset should match both the inertial base shear and the inertial overturning moment. This can be accomplished by a combination of

a) lateral force(s) acting on the hull,

b) lateral force(s) acting equally on all the legs above the upper guide in the direction of the metocean actions, and

c) correcting moment(s) applied as a horizontal or vertical couple(s) to the hull.

The ratio of the total lateral forces acting on the legs above the hull to the lateral forces acting on the hull should not exceed the ratio of the mass of the legs above the upper guide to the total mass of the hull. The moment due to the lateral forces applied to the legs above the upper guide should not exceed the correcting moment required to match the overturning moment, i.e. when applying the forces in b) above, the correcting moment in c) should increase the overturning moment.

Forces or moments due to inertial actions should normally be applied only to structure above the lower guide. Internal leg forces and foundation forces are both important aspects of a site-specific assessment and application of inertial actions to the legs below the lower guide directly affects these in an unrealistic manner.

NOTE The application of the inertial loadset using concentrated forces can result in spurious local stresses.

A.8.8.6 Large displacement effects

There are two displacement effects that should be captured:

— lateral displacement of the hull causes the functional actions to increase global OTM (global P-Δ effects);

— Euler amplification of local member forces increases member stresses (local p-δ effects).

The assessor should be cognisant of how specific software includes these effects. Global displacement effects are normally accounted for as described below. Euler amplification is frequently accounted for in member code checks through use of the member moment amplification factor B_{maf} (see A.12.4). Some methods account for only global effects, while other methods account for both global and local effects.

a) Large displacement methods:

In large displacement methods, the solution is obtained by applying the load case in increments and generating the stiffness matrix for the next load case increment from the deflected shape of the previous increment, iterating on each step if necessary. This method accounts for both global displacement and Euler amplification effects such that $B_{maf} = 1{,}0$ in the moment amplification formulae (see A.12.4).

b) Geometric stiffness methods:

Geometric stiffness methods incorporate a linear correction to the stiffness matrix based on the axial forces present in the elements. It is important that the assessor understand specifically which large displacement effects the software approximates (global and perhaps local) so that the correct value of B_{maf} can be chosen for use in the moment amplification formulae (see A.12.4).

c) Negative spring method:

A simplified geometric stiffness approach allows linear-elastic incorporation of P-Δ effects in an FE program without recourse to iteration. In this approach, a correction term is introduced into the global stiffness matrix prior to analysis. When the analysis is complete, the hull deflections, leg axial forces and leg bending moments include the global P-Δ effects. The derivation of the method is described in ISO/TR 19905-2:2012, A.8.

The correction term is

$$-P_g / L$$

where

P_g is the sum of the leg forces due to functional actions on legs at the hull including the weight of the legs above the hull;

L is the distance from the spudcan reaction point to the hull vertical centre of gravity.

This negative stiffness correction term applied at the hull produces an additional lateral force at the hull proportional to the structural deflection. The resulting (additional) base overturning moment is equal to P_g times the hull displacement.

The negative stiffness is incorporated into the global stiffness matrix by attaching orthogonal horizontal translational spring elements to a node(s) representing the hull centre of gravity. If sets of orthogonal springs are attached to the hull in the vicinity of each leg, using the total spring stiffness divided by the number of legs, the torsional stiffness is also corrected.

If the negative spring(s) are earthed, the additional lateral force (due to the negative stiffness term) causes an overprediction of the horizontal leg reactions. Typically, this is not critical and the horizontal reactions at each leg can be reduced by an amount equal to the force in the spring divided by the number of legs. However, when non-linear foundation elements are used, the earthed-spring approach overpredicts the horizontal foundation reactions and results in erroneous foundation responses. The overprediction of the horizontal leg reactions can be avoided if sets of negative horizontal springs are defined for each leg and connected between the hull and the spudcan.

The application of negative springs to the model accounts for global displacement effects but does not include local Euler effects for individual members; therefore, code checks should include appropriate terms to account for amplification of local moments (see A.12.4).

A.8.8.7 Conductor actions

The conductor actions can be applied as static forces. The reaction due to the tension and hydrodynamic action on the conductor should be included in the jack-up's global analysis model and applied through the support point on the hull.

The effects of stiffness and damping in the conductor are not generally modelled in a jack-up structural assessment because they normally have negligible influence on the global jack-up response.

Structural integrity assessment of an individual conductor is outside the scope of this document.

A.8.8.8 Earthquake actions

See 10.7 and A.10.7 for earthquake actions.

A.8.8.9 Ice actions

See 10.8 and A.10.8 for ice actions.

A.9 Guidance on foundations

A.9.1 Applicability

No guidance is offered.

A.9.2 General

No guidance is offered.

A.9.3 Geotechnical analysis of independent leg foundations

A.9.3.1 Foundation modelling and assessment

A.9.3.1.1 General

In 9.3.1 and A.9.3.1 are addressed the approaches to foundation modelling for

— response analysis; and

— foundation assessment checks.

The response analysis should incorporate dynamic effects using a compatible or conservative foundation model. Dynamic effects can either be applied by means of a set of added inertial actions or be directly included in the analysis. There is a specific set of foundation assessment checks for each of the foundation models that can be selected for the response analysis, as shown in Table A.9.3-1.

The foundations of independent-leg jack-ups approximate large inverted cones, commonly known as spudcans. Roughly circular in plan, spudcans typically have a shallow conical underside (in the order of 15° to 30° to the horizontal) and can have a sharp protruding point. Other spudcan geometries are not uncommon (see Figure A.9.3-1). Large jack-up spudcans can be in excess of 20 m in diameter, with shapes varying with manufacturer and jack-up. Non-circular spudcans can be approximated by means of a disc with equivalent diameter. The foundation capacity formulae given in A.9.3.2 are applicable to circular spudcans. Skin friction on the legs or spudcan is often ignored. Due consideration should be given to the tapered geometry of most spudcans when assessing the foundation capacity.

NOTE Symbols that are not defined in the text can be found in 4.1.5.

Dimensions in metres

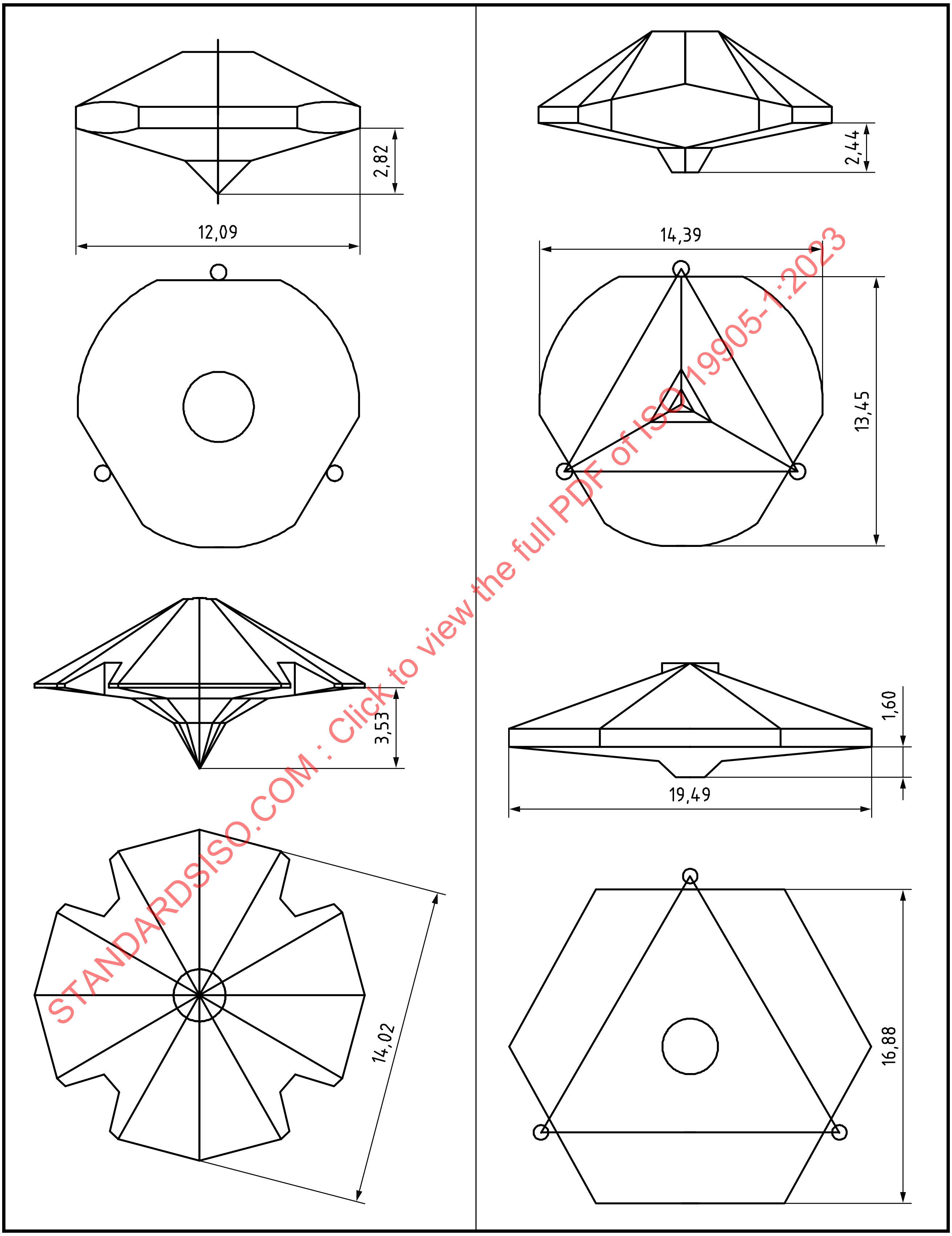


Figure A.9.3-1 — Typical spudcan geometries

A.9.3.1.2 Approaches to foundation assessment

The jack-up and its foundation can be assessed using any of the fixity treatments in Table A.9.3-1. The overall assessment procedure of the jack-up is given in Figure A.10.4-2.

There are certain cases that are not covered in the checks described above, which should be considered separately; some of the more common examples are listed below.

- Cases where the long-term (drained) soil bearing capacity is less than the short-term (undrained) capacity, e.g. for overconsolidated clays or cohesive silts with significant sand seams.
- Cases where a degradation of soil strength occurs due to cyclic loading. This can be of particular significance for silty soils and/or carbonate materials.
- Cases where an increase in spudcan penetration occurs and a potential for punch-through exists, e.g. due to cyclic loading.
- Cases where horizontal seams of weak soil are located beneath the spudcan that can result in inadequate horizontal (sliding) capacity and sliding instability.

If any of the above circumstances exist, further analysis should be carried out.

In the case of partial embedment of a conical spudcan, e.g. in sandy soils, after preloading, additional spudcan embedment can result in a considerable increase in foundation capacity, which can be used in the assessment checks.

In some circumstances, the foundation capacities and stiffnesses are not sufficient for the unit to satisfy the acceptance criteria (Clause 13) based on the applied preload. In such cases, the assessment can be based on foundation capacities and stiffnesses calculated using soil strength parameters and partial material factor γ_m instead of the applied preload in accordance with the approach described in E.4. .

Table A.9.3-1 — Approaches to foundation assessment

Fixity treatment in response analysis		Foundation assessment	Acceptance category	Subclause
Pinned		Simple preload check, Windward leg check (both are subject to limitations)	Level 1; Step 1a Level 1; Step 1b	A.9.3.6.2 A.9.3.6.3
		Bearing and sliding checks using vertical-horizontal capacity envelope	Level 2; Step 2a	A.9.3.6.4
		Displacement check using the vertical-horizontal capacity envelope and load-penetration curve; should also meet the Level 2; Step 2a sliding checks	Level 3; Step 3a	A.9.3.6.6
Fixity	Simple interaction surface (secant model)	Bearing and sliding checks (uses the same procedure as in Level 2; Step 2a)	Level 2; Step 2b	A.9.3.6.5
		Displacement check using the vertical-horizontal capacity envelope and load-penetration curve; should also meet the Level 2 sliding checks	Level 3; Step 3a	A.9.3.6.6
	Full interaction surface (yield interaction model)	Foundation checks are implicit in the non-linear model; should also meet the Level 2 sliding checks unless implicitly included	Level 2; Step 2c or Level 3; Step 3b	A.9.3.6.5 A.9.3.6.6
	Continuum	Foundation checks are implicit in the non-linear model	Level 3; Step 3b	A.9.3.6.6

A.9.3.1.3 Simple pinned foundation

Pinned foundation treatment incorporates a simple preload and sliding check (both subject to limitations). Otherwise a check on foundation capacity in terms of vertical-horizontal capacity and sliding capacity should be performed.

A.9.3.1.4 Linear vertical, linear horizontal and secant rotational stiffness

This foundation fixity treatment incorporates a check on foundation capacity in terms of vertical-horizontal capacity and sliding capacity. The amount of rotational fixity is not directly involved in a checking formula. However, the moment, bearing and sliding interaction is implicitly checked through the use of the yield surface function. Vertical-horizontal and sliding capacities should still be checked explicitly through the procedures described in A.9.3.6.

A.9.3.1.5 Non-linear vertical, horizontal and rotational stiffness

The vertical, horizontal and moment interaction is implicitly checked through the use of the yield interaction model as described in A.9.3.4.2.3. No other checks are required providing that sliding is incorporated in the model.

A.9.3.1.6 Non-linear continuum foundation model

This model should not be used unless one of the simpler analysis methods above has been used to provide a benchmark for the results. The soil model should be capable of capturing the non-linear behaviour for the strain levels expected in the response. The interface between the spudcan and the soil should be modelled to account for effects such as sliding due to insufficient friction.

A.9.3.2 Leg penetration during preloading

A.9.3.2.1 Analysis method

A.9.3.2.1.1 General

The conventional procedure for the assessment of spudcan load/penetration behaviour is given in the following steps.

a) Model the spudcan.

b) Compute the gross ultimate vertical bearing capacity, Q_V, of an open hole for various depths of the bearing area below sea floor using closed form bearing capacity solutions for the best estimate soil strength profile. A low representative value and a high representative value of the soil strength profile should also be used to assess the implications of the range of spudcan penetrations.

c) Use Formula (A.9.3-1) to convert the gross ultimate vertical bearing capacity at each depth to the available structural spudcan reaction, V_L, by deducting, when appropriate, the submerged weight of the backfill, W_{BF}, and adding the soil buoyancy of the spudcan below bearing area, B_S, calculated as $B_S = \gamma' V_D$ as described in A.9.3.2.1.5.

$$V_L = Q_V + B_S \qquad \text{(with no backfill)}$$

$$V_L = Q_V - W_{BF} + B_S \qquad \text{(with backfill)} \qquad \text{(A.9.3-1)}$$

See A.9.3.2.1.4.

NOTE Formula (A.9.3-1) assumes the gross vertical bearing capacity is equal to the vertical spudcan reaction during preloading. Ultimate vertical bearing capacity can exceed preload spudcan reaction, particularly for competent soil conditions.

d) Plot the available structural spudcan reaction, V_L as a curve against penetration, accounting for the distance of the spudcan tip beneath the depth of the bearing area by increasing the penetration used in the capacity calculation by this distance. The curve should extend to a suitable depth beyond the expected penetration. This depth should normally be 1,5 times the expected penetration or to the penetration associated with 1,5 times the preload reaction.

e) Enter the curve of available structural spudcan reaction versus spudcan penetration with the maximum preload reaction at the spudcans and read off the predicted spudcan penetration.

A.9.3.2.1.2 Modelling the spudcan

For conventional foundation analyses, the spudcan can often be modelled as a flat circular foundation. The equivalent diameter is determined from the area of the actual spudcan cross-section in contact with the sea floor, or where the spudcan is fully embedded, from the largest cross-sectional area in plan (see Figure A.9.3-2). Foundation analyses are then performed for this circular foundation at the greatest embedment depth, D_{embed}, of the maximum cross-sectional area in contact with the soil.

Since the depth of spudcan penetration is normally reported and presented as the distance from the spudcan tip to the sea floor, care should be taken to use the appropriate value in the analysis and presentation of results.

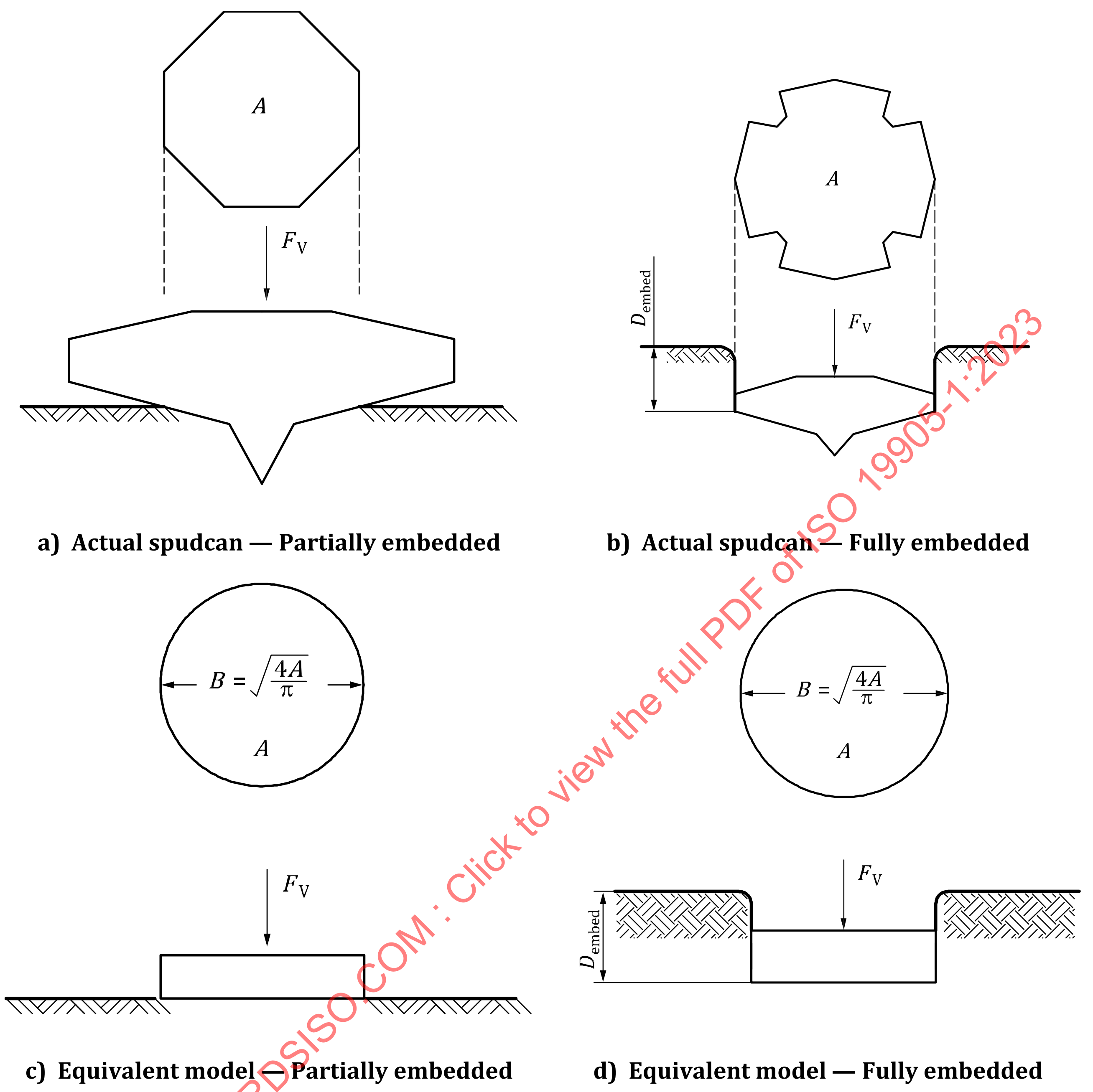
Conical shapes are discussed in Annex E. Other configurations, e.g. rectangular spudcans or legs with significant skin friction, can require alternative treatment.

When a penetration analysis uses bearing capacity factors that account for the conical underside of the spudcan, at each depth the equivalent cone angle (β, Figure A.9.3-3 and Annex E) for the amount of spudcan penetrated should be evaluated. With reference to Figure A.9.3-3, the equivalent cone should be taken such that

— the diameter, B, of the cone at its top gives an area equal to the largest plan cross-sectional area in contact with the soil,

— the cone angle should be determined so as to enclose the same volume as that of the spudcan below the sea floor, and

— once the largest plan area is mobilized, the volume and equivalent cone angle remain constant.



a) Actual spudcan — Partially embedded

b) Actual spudcan — Fully embedded

c) Equivalent model — Partially embedded

d) Equivalent model — Fully embedded

Key

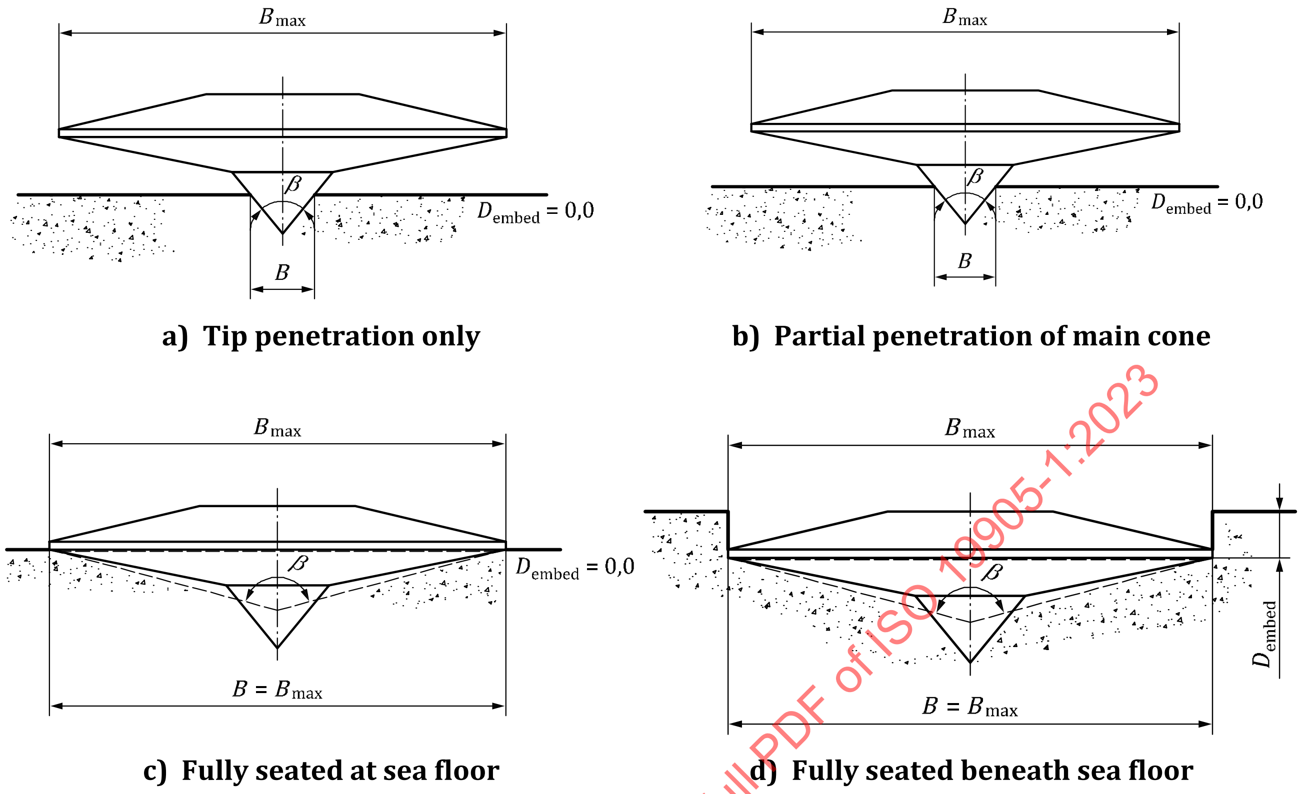
A effective bearing area based on cross-section taken at uppermost part of bearing area in contact with soil

B effective spudcan diameter

D_{embed} greatest embedment depth of maximum cross-sectional spudcan bearing area below the sea floor

F_V gross vertical force acting on the soil beneath the spudcan due to the assessment load case

Figure A.9.3-2 — Spudcan foundation model



Key

B_{max} maximum effective spudcan diameter

B effective spudcan diameter

D_{embed} greatest embedment depth of maximum cross-sectional spudcan bearing area below the sea floor

β effective cone angle

NOTE 1 Effective cone indicated by dashed lines.

NOTE 2 Based on Martin (1994)[132].

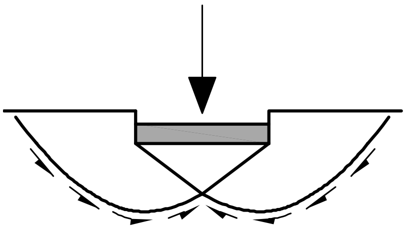
Figure A.9.3-3 — Calculating an equivalent conical spudcan for various embedments

A.9.3.2.1.3 Modelling the soil

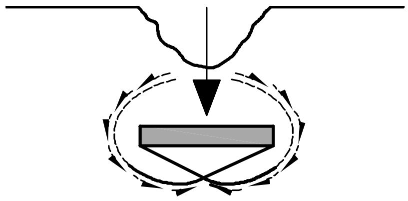
The soil beneath the spudcan fails as the foundation is loaded during preloading until equilibrium is achieved at the end of the preloading operation. Figure A.9.3-4 shows different failure mechanisms for various soil conditions, which range from conventional bearing capacity failure in uniform soils, potential punch-through for layered soils, squeezing, and combinations of all of these mechanisms. The soil model should be sufficiently accurate to represent the behaviour of spudcan and soil characteristics during preloading.

An appropriate soil model should be used for layered soils to account for the effects of punch-through or squeezing, e.g. local failure of a weak layer between two stronger layers. It is mentioned that an artificial punch-through condition can be created as a result of soil consolidation occurring during pauses in leg penetration whilst the spudcan is loaded to less than full preload. Such pauses can occur during installation operations or geotechnical investigation from a jack-up prior to full preloading.

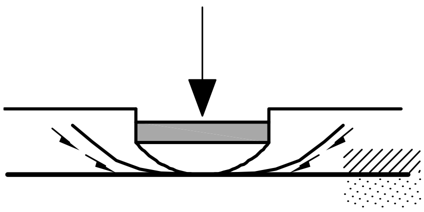
The analysis methods in A.9.3.2.1.4 to A.9.3.2.6.6 address the failure mechanisms shown in Figure A.9.3-4.



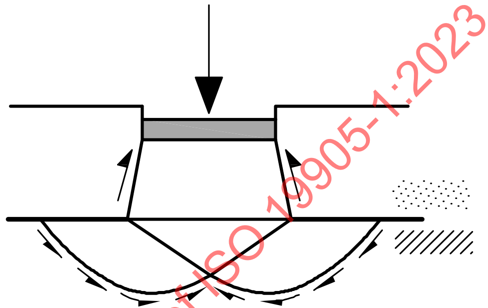
a) Conventional bearing capacity failure: uniform soil



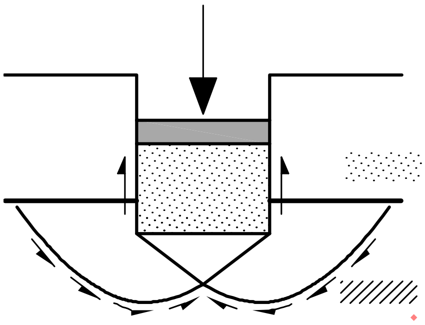
b) Deep bearing capacity: uniform soil



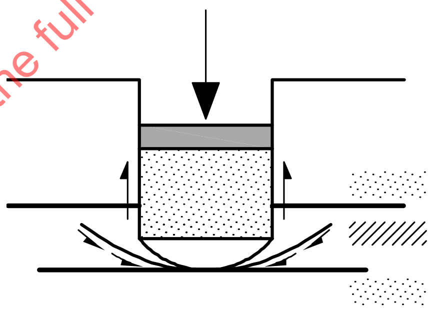
c) Squeezing



d) Punch-through



e) Punch-through (with trapped soil plug)



f) Punch-through (with trapped soil plug) and squeezing

Figure A.9.3-4 — Spudcan bearing failure mechanisms

A.9.3.2.1.4 Backfill

With reference to Figure A.9.3-5, soil backfill on top of the spudcan can result from backflow or infill. Regardless of the mechanism, this soil,

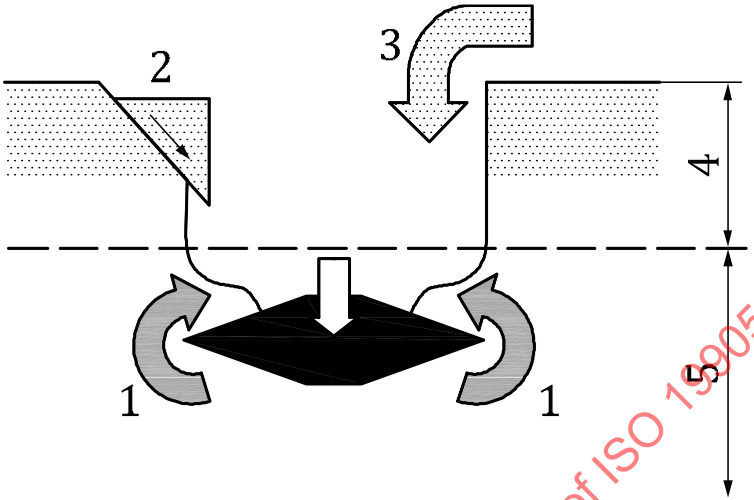
a) increases penetration if it occurs during preloading,

b) reduces capacity available to support downward structural loads at the spudcan if it occurs after preloading, and

c) always increases the uplift resistance.

Backflow is the soil that flows from beneath the spudcan, around the sides, and onto the top and is more likely to occur in clays than in sands. Backflow can occur at shallow penetrations, but is more likely to occur at deeper penetrations. In very soft clays, complete backflow is likely to occur. In firm to stiff clays and granular materials, where spudcan penetration is expected to be small, the possibility of backflow diminishes. In general, backflow due to additional penetration during elevated operations is not expected to occur. If it is predicted, the effects should be taken into account.

Infill is the soil on top of the spudcan that results from cavity wall collapse or sediment transport, e.g. where there is a sand veneer over clay. Cavity wall collapse can occur during or after preloading; sediment transport is only of significance after preloading. Cavity wall collapse can occur slowly or suddenly. If it occurs suddenly during preloading, it can cause a rapid increase in penetration.



Key

1 backflow

2 infill–- wall failure

3 infill–- sediment transport

4 region subject to infill processes

5 region subject to backflow

NOTE Backfill includes backflow and infill.

Figure A.9.3-5 — Backflow and infill

The submerged weight of backfill ($W_{BF,o}$) during preloading loads the top of the spudcan and results in additional penetration.

Backfill that occurs after preload has been applied and held ($W_{BF,A}$) provides additional weight on the spudcan. This backfill reduces the vertical reaction that the foundation can support to resist the overturning moment. Conversely, any subsequent backfill increases the available uplift capacity of the windward leg(s).

The minimum value of the backfill weight due to backflow during preloading, $W_{BF,omin}$, depends on the limiting depth of cavity, H_{cav}, that remains open above the spudcan during penetration as given in Formula (A.9.3-2):

$$W_{BF,omin} = \gamma'\left[A(D_{embed} - H_{cav}) - \left(V_{spud} - V_D\right)\right] \quad \text{(with backflow, i.e. } W_{BF,omin} \text{ always positive)}$$

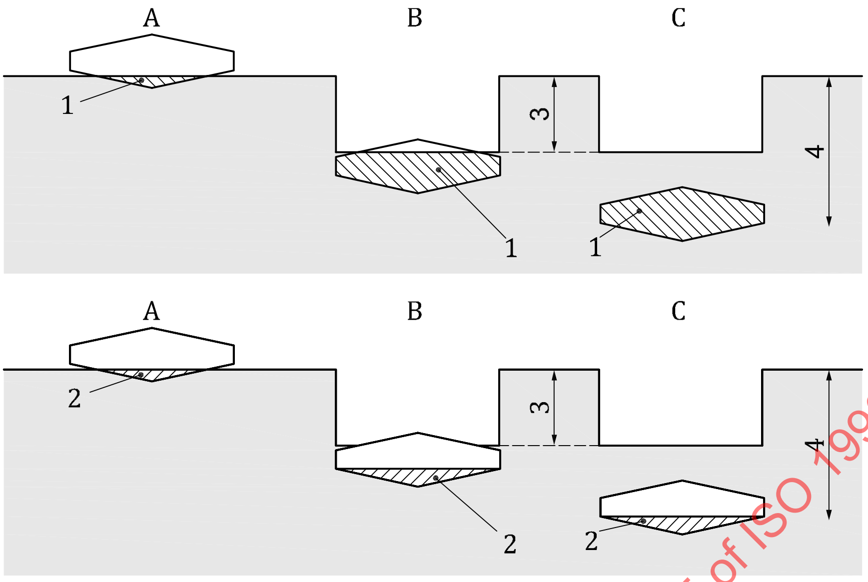
$$W_{BF,omin} = 0 \quad \text{(with no backfill)} \qquad \text{(A.9.3-2)}$$

where

V_{spud} is the total volume of the spudcan beneath the backfill;

V_D is the volume of the spudcan below the maximum bearing area that is penetrated into the soil, refer to Figure A.9.3-6; V_D is zero for a flat-based spudcan.

Care should be taken when calculating V_{spud} when the spudcan is not fully covered with backflow material; refer to Figure A.9.3-6.



Key

A partial spudcan penetration

B full spudcan penetration with partial backfill, $W_{BF,o}$ during penetration

C full spudcan penetration with full backfill, , $W_{BF,o}$ during penetration

1 the total volume of the spudcan below the backfill, V_{spud}

2 the volume of the spudcan below the maximum bearing area that is penetrated into the soil, V_D

3 depth of cavity that remains open above spudcan, H_{cav}

4 greatest embedment depth, D_{embed}, of maximum cross-sectional spudcan bearing area below the sea floor

Figure A.9.3-6 — Definition of spudcan volumes

For a single-layer clay with uniform shear strength or shear strength increasing with depth at a rate, ρ, Formula (A.9.3-3) from Hossain and Randolph (2009a)[94] can be used to estimate H_{cav}. This expression and the supporting data are graphically presented in Figure A.9.3-7. Formula (A.9.3-4) from Hossain and Randolph (2009a)[94] can be used to estimate H_{cav} for multi-layer clays with moderate changes of strength, iterating to establish consistent values for H_{cav}/B and s_{uH}.

$$H_{cav} / B = S^{0,55} - 0,25S \quad \text{(A.9.3-3)}$$

$$H_{cav} / B = [s_{uH} / (\gamma' B)]^{0,55} - 0,25[s_{uH} / (\gamma' B)] \quad \text{(A.9.3-4)}$$

where

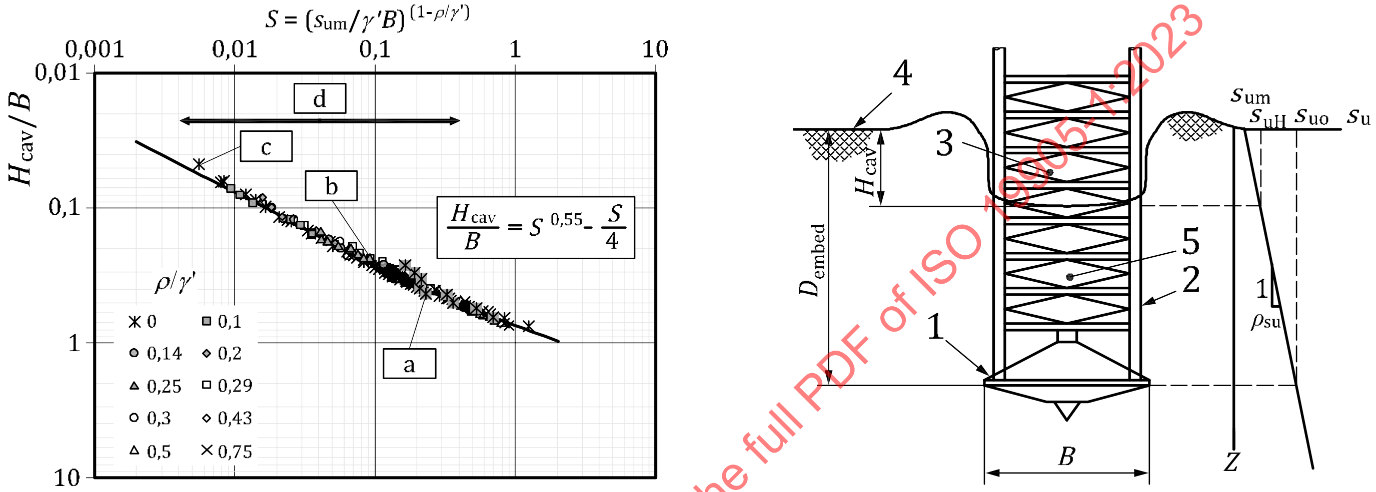
$$S = \left(\frac{s_{um}}{\gamma' B}\right)^{\left(1 - \frac{\rho}{\gamma'}\right)} \quad \text{(A.9.3-5)}$$

s_{uH} is the undrained shear strength at a depth of H_{cav} below sea floor;

s_{um} is the undrained shear strength at the sea floor.

The onset of backflow marks the transition between shallow and localized failure mechanisms. In addition to affecting the vertical reaction beneath the spudcan during preloading, the degree of backflow influences the embedment condition of the spudcan and, hence, the uplift resistance (see A.9.4.5), horizontal and moment restraint and, therefore, the yield surface (see A.9.3.3.3).

In silica sand, it is unusual for a conical spudcan to penetrate beyond its widest point. However, if this is predicted, the potential for soil infilling on top of the spudcan should be considered during preloading (as the soil assumes its angle of repose).



a) Experimental data and curve-fit

b) Idealized scenario

Key

1 spudcan

2 leg truss

3 cavity

4 sea floor

5 soil backflow

B effective spudcan diameter (typically 11 m to 20 m)

D_{embed} greatest depth of embedment of maximum cross-section in contact with the soil

H_{cav} limiting depth of cavity that remains open above the spudcan during penetration

s_{uH} undrained shear strength at base of cavity

s_{um} undrained shear strength at sea floor

s_{u0} undrained shear strength at depth of maximum spudcan bearing area

s_u undrained shear strength

Z depth below sea floor

γ' submerged unit weight of soil

ρ_{su} rate of increase in undrained shear strength with depth

a Centrifuge test data.

b Large deformation FE analyses: non-uniform strength.

c Large deformation FE analyses: uniform strength.

d Typical design range.

Figure A.9.3-7 — Estimation of limiting cavity depth, H_{cav}, due to backflow during installation

A.9.3.2.1.5 Required bearing capacity

At maximum preload, the initial gross ultimate bearing capacity, Q_{Vo}, under the spudcan is equal to the preload reaction, V_{Lo}, plus the submerged weight of any backfill onto the spudcan, less the soil buoyancy of the spudcan below the bearing area as given in Formula (A.9.3-6):

$$Q_{Vo} = V_{Lo} + W_{BF,o} - B_S \tag{A.9.3-6}$$

where

$W_{BF,o}$ is the submerged weight of the overburden on top of the spudcan from backfill during preloading, which is not less than $W_{BF,omin}$;

$B_S = \gamma' V_D$ is the soil buoyancy of spudcan below bearing area, i.e. the submerged weight of soil displaced by the spudcan below D_{embed}, the greatest depth of embedment of the maximum cross-sectional spudcan bearing area below the sea floor;

V_D is the volume of the spudcan below the lowest level of maximum bearing area that is penetrated into the soil; V_D is zero for a flat-based spudcan.

The initial gross ultimate vertical bearing capacity, Q_{Vo}, is established by preload operations and related to V_{Lo}. However, in some cases, subsequent actions can cause further penetration and a corresponding increase in Q_V, as is consistent with the load-penetration formulae given in A.9.3.2.2 through A.9.3.2.6.

A.9.3.2.2 Penetration in clays

The gross ultimate vertical bearing capacity of a foundation in clay of uniform shear strength (undrained failure in clay, $\phi = 0°$) at a specific depth can be expressed as given in Formula (A.9.3-7):

$$Q_V = (s_u N_c s_c d_c + p'_o)\pi B^2/4 \tag{A.9.3-7}$$

where

p'_o is the effective overburden pressure at the greatest embedment depth, D_{embed}, of the maximum bearing area;

d_c is the bearing capacity depth factor, $d_c = 1 + 0{,}2\ (D_{embed}/B) \leq 1{,}5$.

For circular footings, the product $N_c \cdot s_c$ should be taken as 6,0.

For the selection of the design undrained shear strength s_u, an evaluation should be made of the sampling method, the laboratory test type and the field experience regarding the prediction and observations of spudcan penetrations.

Traditionally, the value of N_c has been determined from solutions for strip footing on homogeneous clay, with shape and depth factors based on Skempton (1951)[169]. However, these factors are significantly affected by the gradient of shear strength with depth [see Young et al. (1984)[212] and Houlsby and Martin (2003)[99]].

Theoretical solutions for circular conical foundations on clays of uniform and increasing strength with depth have been provided by Houlsby and Martin (2003)[99], as presented in E.1. The solutions give a theoretical lower bound to the soil resistance and should, therefore, provide an upper bound prediction of penetration.

The total bearing capacity factors for rough spudcans, modelled as rough circular plates, are given in Table A.9.3-2. Further bearing capacity factors are given in E.1 for the following parameter ranges (see Figures A.9.3-2, A.9.3-3 and A.9.3-7):

- cone angles β between 60° and a flat plate of 180°;
- embedment depths, D_{embed}, between 0 and 2,5B;
- values of shear strength gradient $\rho_{su}B/s_{um}$ between 0 and 5, where ρ_{su} is the rate of increase in undrained shear strength with depth, from a value of s_{um} at the sea floor.

NOTE For soil layers that do not extend to the sea floor surface, sum refers to the undrained shear strength at the top of the layer.

The tables in Annex E provide a theoretical lower bound to the total bearing factor $N_c \cdot s_c \cdot d_c$ to apply to the shear strength at the spudcan base level, s_{uo}, for the full range of the above parameters. Alternatively, Houlsby and Martin (2003)[99] indicates that using the shear strength, s_u, at a depth of 0,09B below the spudcan base level together with the bearing factors given in Table A.9.3-2 for a foundation on uniform strength clay provides answers that are within ± 12 % of the theoretical lower bound solutions.

Alternatively, field experience in the Gulf of Mexico (Young, 1984)[212] indicates that for typical Gulf of Mexico shear strength gradients and spudcan dimensions, spudcan penetrations in clay are well predicted by selecting s_u as the average over a depth of $B/2$ below the widest cross-section in combination with the bearing capacity and simplified depth factor formula from Skempton (1951)[169] provided in Formula (A.9.3-7). A comparison was made Menzies and Roper (2008)[138] between measured load-penetration records from thirteen Gulf of Mexico clay sites with linearly increasing shear strength profiles and spudcan penetration predictions from four bearing capacity formulations, namely Skempton (1951)[169], Hansen (1970)[88], Houlsby and Martin (2003)[99] and Hossain et al. (2006)[93]. The comparisons indicate that the Houlsby and Martin method provides a good lower bound load-penetration prediction indicating deeper penetrations, the Hossain et al. method provides an upper bound load-penetration prediction, usually predicting shallower penetrations than measured, and the Skempton and Hanson bearing capacity factors provide reasonable predictions of average penetrations. The Hossain et al. (2006)[93] bearing capacity method was modified (Hossain et al. 2009)[96] to provide a load-penetration prediction method that accounts for soil strain rate dependency and strain softening during spudcan penetration. Hossain et al. (2014)[97] compared load-penetration results from the modified bearing capacity model with data from Menzies & Roper (2008)[138] and found good agreement.

Some jack-up rigs with more than 3 legs are able to apply the pre-drive in minutes; conventional water ballast preload operations take hours or days to complete. The bearing capacity when pre-driving should utilize bearing capacity methods that allow for changes in the shear strength due to strain rate effects. Research performed by Hossain and Randolph (2009c)[96] developed a bearing capacity model that includes the effect of strain rate dependency to predict load-penetration. A discussion of strain rate effects on the prediction of load-penetration is also presented in Versteele et al. (2017)[195].

For clay layers with distinct strength differences, methods for layered soils should be used; see A.9.3.2.6.

Table A.9.3-2 — Bearing capacity factors for rough circular plate on homogeneous clay (Houlsby and Martin, 2003[99])

Embedment ratio, D_{embed}/B	Bearing factor, $N_c \cdot s_c \cdot d_c$
0	6,0
0,1	6,3
0,25	6,6
0,5	7,0
1,0	7,7
≥ 2,5	9,0

The bearing factor is nonlinear with respect to the embedment ratio. It is necessary to use caution when estimating an appropriate bearing factor for embedment ratios other than those given in Table A.9.3-2.

A.9.3.2.3 Penetration in soils with partial drainage

It is recommended that analyses for drained conditions (modelled as sand) and undrained conditions (modelled as clay) be performed to estimate the range of penetrations.

Partial drainage conditions and penetration in soils can be assessed using the approaches described by Finnie and Randolph (1994b)[72] and Erbrich (2005)[70]; the latter reference also describes the use of cyclic preloading in silts.

A.9.3.2.4 Penetration in silica sands

Spudcan penetration in silica sand is usually analysed as a drained process, in which no excess pore water pressure is generated. In drained conditions, the gross ultimate vertical bearing capacity of a circular foundation in homogeneous frictional material can be expressed as given in Formula (A.9.3-8):

$$Q_V = \frac{\gamma' d_\gamma N_\gamma \pi B^3}{8} + \frac{p_o' d_q N_q \pi B^2}{4} \qquad \text{(A.9.3-8)}$$

where

d_γ is the depth factor on self weight for drained soils, $d_\gamma = 1{,}0$;

d_q is the depth factor on surcharge for drained soils, $d_q = 1 + 2\tan\phi'\ (1\text{-}\sin\phi')^2 \arctan(D_{embed}/B)$ where $\arctan(D_{embed}/B)$ is in radians and $\phi'$is the effective angle of internal friction for sand in degrees;

B is the effective spudcan diameter in contact with the soil;

γ' is the submerged unit weight of the soil;

N_γ and N_q are dimensionless bearing capacity factors calculated for the axisymmetric case (no further shape factor should be applied).

If the spudcan penetrates beyond its widest point, the overburden of soil above this point creates an effective surcharge, p_o', at the level of the widest point, which leads to additional bearing capacity.

Theoretical values of N_γ and N_q calculated using the slip-line method for a flat, rough circular footing in Martin (2003)[133] are given in Table A.9.3-3 for soil friction angles from 20° to 40°. These N_γ and N_q factors can also be applied to (blunt) conical spudcans that are not fully rough, since the error involved is generally small compared with that arising from the uncertainty in selecting the soil friction angle; for example, Table A.9.3-3 shows that a 1° change in ϕ' gives at least a 20 % change in N_γ A more detailed penetration analysis can be performed using the values of N_γ for conical footings tabulated in Annex E; these cover a range of cone apex angles and interface roughness coefficients.

Adequate consideration should be given to the selection of an appropriate soil friction angle (see E.2).

Table A.9.3-3 — Bearing capacity factors for a flat, rough circular footing (Martin, 2003)[133]

Friction angle ϕ' Degrees	Bearing factor N_γ	Bearing factor N_q
20	2,4	9,6
21	2,9	10,9
22	3,5	12,4
23	4,2	14,1
24	5,1	16,1
25	6,1	18,4
26	7,3	21,1
27	8,8	24,2
28	10,6	27,9
29	12,8	32,2
30	15,5	37,2
31	18,8	43,2
32	22,9	50,3
33	27,9	58,7
34	34,1	68,7
35	41,9	80,8
36	51,6	95,4
37	63,7	113,0
38	79,1	134,4
39	98,7	160,5
40	123,7	192,7

A.9.3.2.5 Penetration in carbonate sands

A.9.3.2.5.1 General

Penetrations in carbonate sands are highly unpredictable and can be minimal in strongly cemented materials, or large, in uncemented materials. Cementation, crushable particles, high in situ void ratios and compressibility are some of the characteristics of calcareous sediment that have led to the conclusion that the routine bearing capacity methods linked to the frictional soil strength are inappropriate [Poulos and Chua (1985)[148], Le Tirant and Nauroy (1994)[121] and Finnie and Randolph (1994a)[72]]. Extreme care should be exercised when operating in these materials.

A.9.3.2.5.2 Uncemented carbonate materials

Relatively large spudcan penetrations have been reported for uncemented carbonate materials despite high laboratory friction angles [Dutt and Ingram (1988)[66]]. This can be attributed to either the high compressibility of these materials or low shear strengths due to high voids ratio and a collapsible structure.

The leg penetration is governed by both the strength and deformation characteristics of the soils. The compressibility of carbonate sands is relatively higher than that of silica sands. Hence, greater penetrations should be expected for carbonate sands relative to silica sands despite the similar or even higher laboratory friction angles. This is supported by both experimental studies [Poulos and Chua (1985)[148], Pan (1999)[144], Pan et al. (1999)[145], and Byrne and Houlsby (2001)[39]] and theoretical studies [Yeung and Carter, (1989)[209]] on model foundations.

A.9.3.2.5.3 Cemented carbonate materials

Natural cementation in calcareous sediments is formed by carbonate precipitation. Model spudcan experiments on artificially cemented calcareous soils have shown that the pure vertical bearing response of circular foundations can also be described as bi-linear, with a yield point that is similar to the yield stress in 1-dimensional compression [Poulos and Chua (1985)[148], Houlsby et al. (1988)[100], Sharp and van Seters (1988)[166], and Randolph and Erbrich (1999)[153]]. The bearing resistance then increases with continuing displacements, with no clear failure point. This behaviour is consistent with local or punching shear failure. Randolph and Erbrich (1999) [153] explain this bi-linear shape as being attributable to the very small settlement expected before the yield pressure is exceeded.

A.9.3.2.5.4 Predictive methods

The predictions of spudcan penetrations in carbonate sands are likely to be less accurate than those for silica sands because carbonate sands generally have high porosity and a varying degree of cementation.

Spudcan penetration occurs due to a combination of soil compression and soil failure. The use of the conventional general shear failure model for sand for predicting the penetration is, therefore, not appropriate. This model is, however, generally adopted for penetration predictions in carbonate sands but requires a careful assessment of the friction angle. The reduction of the friction angles is typically in the range of 3° to 7° for cemented and uncemented carbonate sands.

Special attention is required for sites with a stronger cemented soil layer overlying weak, uncemented layers with careful consideration given to the type of punch-through mechanism.

Randolph et al. (1993)[154] and Finnie and Randolph (1994a)[71] outline a bearing modulus method for uncemented calcareous sands. This is based on the results of a series of centrifuge experiments of model footings that indicate that the vertical bearing capacity increased linearly with depth. An estimation of the bearing pressure can be performed as a function of the overburden pressure rather than the self-weight as given in Formula (A.9.3-9):

$$q_u = \gamma' z N_q \qquad \text{(A.9.3-9)}$$

where z is the penetration and N_q is the bearing capacity factor. Whilst $N_q \approx 50$ was found to provide reasonable predictions of the centrifuge test data, it can overpredict the foundation bearing capacity of spudcans in uncemented carbonate soils. Formula (A.9.3-9) can be adapted to calculate the vertical bearing capacity for a conical spudcan by sub-dividing the spudcan geometry vertically into a number of equivalent circular footings as shown in Figure A.9.3-8. The bearing capacity of the area at the base of each slice in contact with the soil can be summed to calculate iteratively the overall bearing capacity of the conical footing for different footing penetrations.

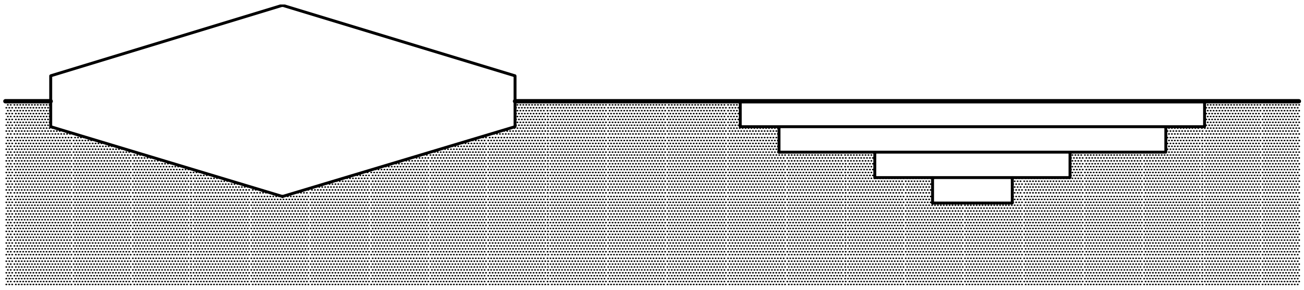


Figure A.9.3-8 — Representation of a conical spudcan by equivalent circular footing “slices” for the calculation of vertical bearing capacity in carbonate sands

Other predictive methods for circular spudcans on both cemented and uncemented calcareous sands have been published, including Islam (1999)[107], Islam et al. (2001)[109], Houlsby et al. (1988)[100], Randolph et al. (1993)[154], Finnie and Randolph (1994a)[71], and Yamamoto et al. (2008) [207], (2009)[208]. In concluding that the bearing response of shallow foundations on calcareous sands is better modelled with a compressional deformation mechanism and the punching shear pattern, Yamamoto et al. (2008[207], 2009 [208]) provide simple formulae for the response of shallow footings on compressible sands.

A.9.3.2.6 Penetration in layered soils

A.9.3.2.6.1 General

Three different foundation failure mechanisms should be considered when making spudcan predictions in layered soils:

a) general shear;

b) squeezing;

c) punch-through.

The first failure mechanism occurs if soil strengths of subsequent layers do not vary significantly. Thus, an average soil strength (either s_u or ϕ') can be determined below the spudcan. The spudcan penetration versus foundation capacity relationship is then generated using criteria from A.9.3.2.2 to A.9.3.2.5.

Criteria for the other two failure mechanisms (squeezing and punch-through) are given in A.9.3.2.6.2 to A.9.3.2.6.6. Punch-through is of particular significance since it concerns a potentially dangerous situation where a strong layer overlies a weak layer and, hence, a small additional spudcan penetration can be associated with a significant reduction in vertical bearing capacity that results in rapid leg penetration.

Backflow and infill should be considered.

A.9.3.2.6.2 Squeezing of clay

Squeezing failure of a soft clay layer overlying a significantly stronger layer (see Figure A.9.3-4 and Figure A.9.3-9) occurs when the thickness of the soft clay beneath the spudcan is less than that required for the general bearing capacity failure mode to apply. In such cases the soft layer squeezes and the vertical foundation capacity is greater than the vertical foundation capacity given by general failure in the soft clay layer, but less than the vertical foundation capacity given by general failure in the underlying significantly stronger layer.

The gross ultimate vertical bearing capacity of a spudcan on a clay layer undergoing squeezing failure can be analysed by methods given by Brown and Meyerhof (1969)[35] and by Vesic (1975)[196] in combination with the bearing capacity and depth factors given by Skempton (1951)[169] as given in Formula (A.9.3-10).

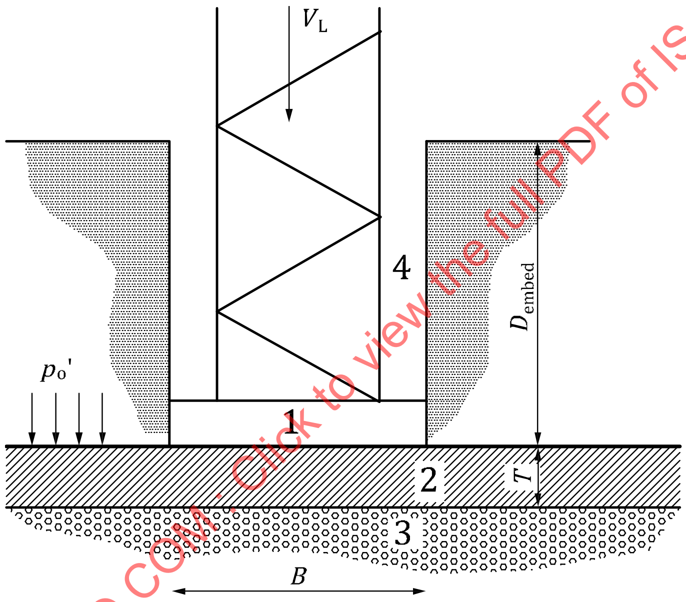
$$Q_v = A\left\{\left(a_s + \frac{b_s B}{T} + \frac{1{,}2D_{\text{embed}}}{B}\right) s_u + p_o'\right\} \geq A\{N_c\, s_c\, d_c\, s_u + p_o'\} \qquad \text{(A.9.3-10)}$$

where the following squeezing factor constants are recommended:

$a_s = 5$

$b_s = 1/3$

and s_u is the undrained shear strength of the soft clay layer.



Key

1 spudcan with effective bearing area, *A*
2 softer clay layer with shear strength, s_u
3 stronger soil
4 no backflow and no infill (i.e. no backfill)
B effective spudcan diameter
D_{embed} embedment depth of spudcan effective bearing area, *A*, below sea floor
V_L available spudcan reaction; see Formula (A.9.3-1)
p'_o effective overburden pressure at depth, D_{embed}
T thickness of weaker clay layer beneath the spudcan effective bearing area

Figure A.9.3-9 — Spudcan bearing capacity analysis — Squeezing clay layer

The squeezing vertical foundation capacity given by Formula (A.9.3-10) should be limited such that it does not exceed the ultimate bearing capacity of the underlying strong soil layer (for $T << B$). Note that as T increases Formula (A.9.3-10) can give a value of Q_V that is less than the corresponding value for the general shear bearing capacity in the soft clay layer. In such cases, Q_V should be taken as the latter.

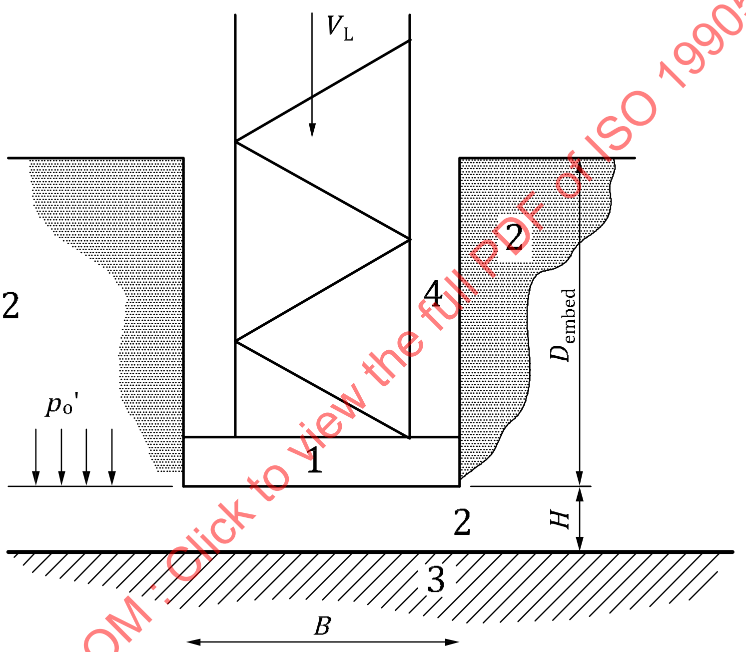
Re-arranging Formula (A.9.3-10) using $s_c N_c$ = 6,0 for the bearing capacity factor at the surface in combination with the expression for the depth factor, d_c, given in A.9.3.2.2 indicates that squeezing failure will occur if $T \leq B/3$, subject to $D_{embed}/B \leq 2{,}5$.

A.9.3.2.6.3 Punch-through: two clay layers

The gross ultimate vertical bearing capacity of a spudcan on the surface of a strong clay layer overlying a weak clay layer can be computed according to Brown and Meyerhof (1969)[35] as given in Formula (A.9.3-11); (see Figure A.9.3-10):

$$Q_V = A\ [3\frac{H}{B}s_{u,t} + N_c s_c (1 + 0{,}2\frac{D+H}{B})s_{u,b} + p'_o] \leq A(N_c s_c d_c s_{u,t} + p'_o) \quad \text{(A.9.3-11)}$$

Formula (A.9.3-11) applies to clay layers of uniform undrained shear strengths.



Key

1 spudcan with effective bearing area, A

2 stronger clay layer with shear strength, $s_{u,t}$

3 weaker clay layer with shear strength, $s_{u,b}$

4 no backflow and no infill (i.e. no backfill)

B effective spudcan diameter

D_{embed} Embedment depth of spudcan effective bearing area, A, below sea floor

V_L available spudcan reaction; see Formula (A.9.3-1)

p'_o effective overburden pressure at embedment depth, D_{embed}

H thickness of stronger clay layer beneath the spudcan effective bearing area

Figure A.9.3-10 — Spudcan bearing capacity analysis — Two clay layers

A.9.3.2.6.4 Punch-through — Sand overlying clay

The gross ultimate vertical bearing capacity of a spudcan on a sand layer overlying a weak clay layer can be computed using a load spread model (see Figure A.9.3-11). In this model, the bearing capacity of the spudcan, Q_V, is calculated by considering a fictitious footing at the interface between the sand and clay layers. Be aware that this is a convenient method for expressing the bearing capacity of the spudcan within the layered soil profile and is not a representation of the actual “punching shear” failure mechanism.

The fictitious footing has an equivalent diameter is as given in Formula (A.9.3-12):

$$B' = B + 2H/n_s \qquad (A.9.3\text{-}12)$$

For sand overlying clay, a load spread factor, n_s, of 3 (see Figure A.9.3-11) has been recommended by Young and Focht (1981)[211] for jack-up foundations. However, comparison with model test data [Jacobsen et al. (1977)[110], Higham (1984)[92], and Craig and Chua (1990a)[50]] suggests a range of n_s from 3 to 5. Conversely, actual spudcan penetration data are available that suggest smaller n_s values (Baglioni, 1982)[21]. It is, therefore, recommended that load spread factors in the range of 3 to 5 be used, consistent with current industry practice.

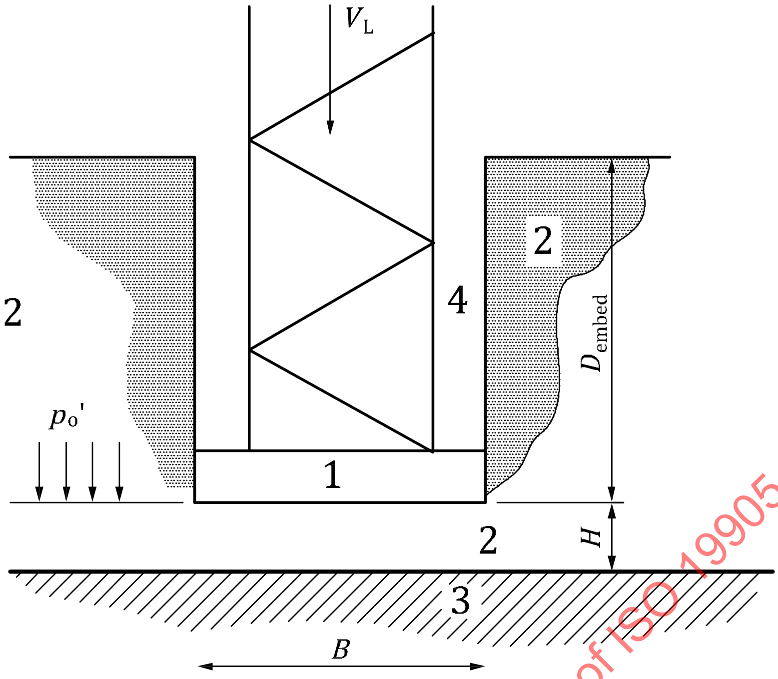
The calculation of the bearing capacity of the fictitious footing should include consideration of the weight of the sand, W, above the fictitious footing at the surface of the lower (clay) layer, based on the assumption of an open cavity being present above the footing, as given in Formula (A.9.3-13):

$$W = 0{,}25\pi\left[B'^2(D_{\text{embed}} + H) - B^2 D_{\text{embed}}\right]\gamma' \qquad (A.9.3\text{-}13)$$

The total capacity is, therefore, as given in Formula (A.9.3-14):

$$Q_V = Q_{u,b} - W \qquad (A.9.3\text{-}14)$$

where $Q_{u,b}$ is the ultimate vertical foundation bearing capacity for the fictitious footing at the interface between the sand and clay layers with no backfill, which can be calculated using Formula (A.9.3-7).



Key

1	spudcan with effective diameter, B
2	sand layer with submerged unit weight of γ'
3	clay layer
4	void above spudcan, i.e. no backflow and no infill (i.e. no backfill)
5	fictitious spudcan with effective diameter, B', at the interface between the upper and lower layers
D_{embed}	embedment depth of actual spudcan below the sea floor
V_L	available spudcan reaction; see Formula (A.9.3-1)
H	distance from spudcan to clay layer below
n_s	load spread factor for sand overlying clay (typically 3 to 5)
p'_o	effective overburden pressure at depth D_{embed}

Figure A.9.3-11 — Spudcan bearing capacity analysis — Sand over clay

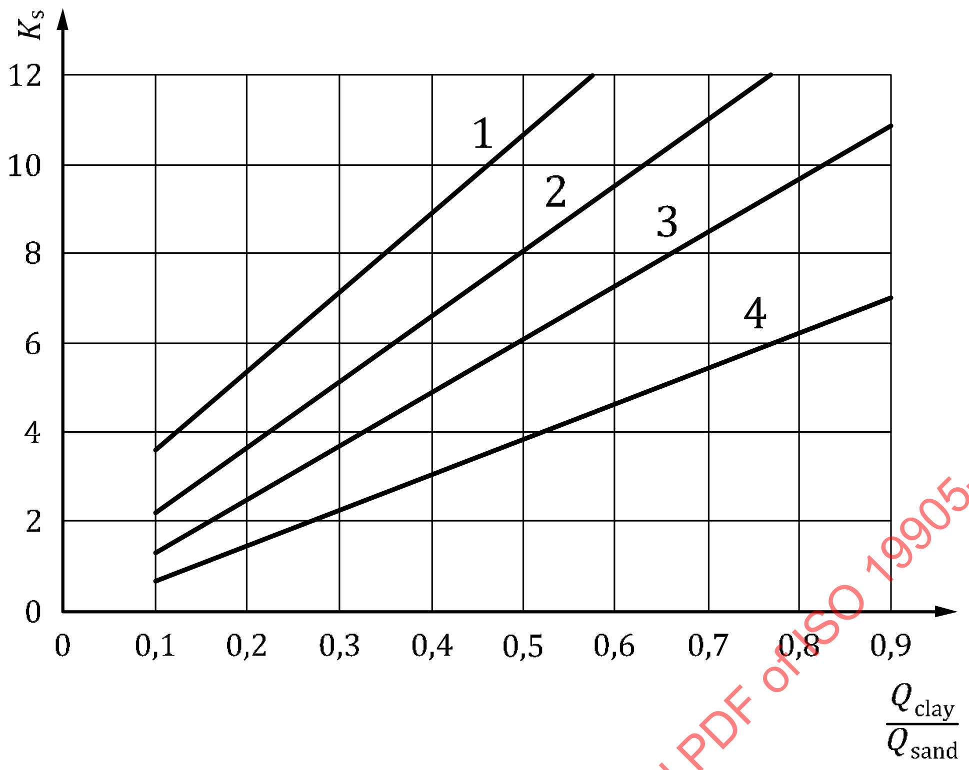
Alternatively, the gross ultimate initial bearing capacity may be calculated using Formula (A.9.3-15) derived from Hanna and Meyerhof (1980)[86]:

$$Q_V = Q_{u,b} - AH\gamma' + 2AH(H\gamma' + 2p'_o)\, K_s \frac{\tan(\varphi)}{B} \tag{A.9.3-15}$$

where $Q_{u,b}$ is determined according to A.9.3.2.2, assuming that the spudcan bears on the surface of the lower clay layer with no backfill.

The punching shear coefficient, K_s, depends on the strength of both the sand layer and the clay layer, which can be derived from the graphs in the reference paper, Hanna and Meyerhof (1980)[86]; see Figure A.9.3-12.

The bearing capacity for Q_{clay} / Q_{sand} ratios less than 0,1 may be calculated using the methods described in either A.9.3.2.6.4 or E.3.



Key

1 $\phi' = 40°$

2 $\phi' = 35°$

3 $\phi' = 30°$

4 $\phi' = 25°$

K_S coefficient of punching shear

Q_{clay} bearing capacity of clay for a surface strip footing of width equal to the spudcan diameter, B

Q_{sand} bearing capacity of sand for a surface strip footing of width equal to the spudcan diameter, B

ϕ' effective angle of internal friction for sand in degrees

Figure A.9.3-12 — Bearing capacity ratio versus coefficient of punching shear for spudcans

An approach based on a centrifuge study has been proposed by Teh et al. (2010)[184]. The load-penetration curve typical of the punch-through condition is represented by a simplified profile consisting of three bearing capacities, namely bearing capacity at sea floor, Q_0 (at $d = 0$), maximum bearing capacity, Q_{peak} (at $d = d_{crit}$), and bearing capacity in the underlying clay (for $d \geq H$). A brief description of the approach is provided in E.3.

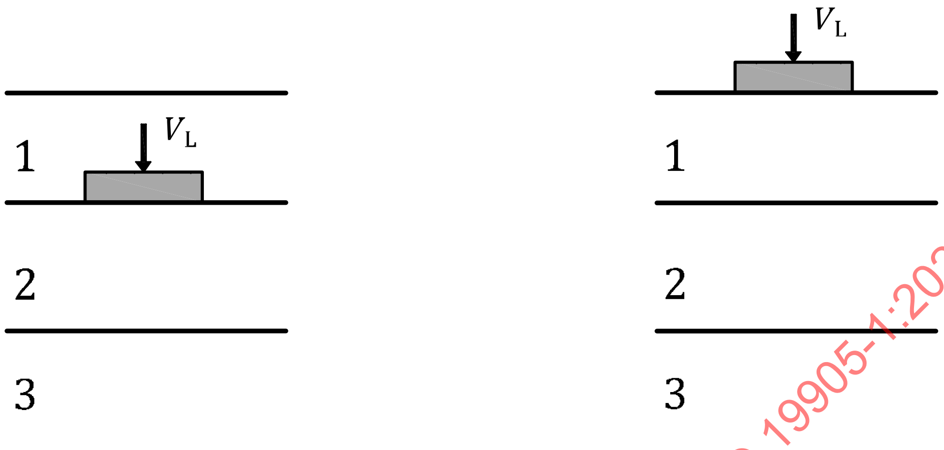
A.9.3.2.6.5 Punch-through — Cemented crust over weak soil

The occurrence of a cemented crust overlying a weak layer of clay or loose sand/silt should be carefully considered. The analysis relies on accurate information on the thickness and strength of the crust and the strength of the underlying layer. The analysis can be performed using simplified load spread models or advanced numerical models. The potential for punch-through can be significantly affected by the shape of the spudcan and its tip.

A.9.3.2.6.6 Three layered systems

The gross ultimate vertical bearing capacity of a spudcan at the top of a three soil layer system can be computed using the squeezing and punch-through criteria for two layer systems. Firstly, the bearing capacity of a spudcan with diameter B at the top of the lower two layers (layers 2 and 3 in Figure A.9.3-

13) is computed. These two layers can then be treated as one (lower) layer in a subsequent two layer system analysis involving the upper layer (layer 1 in Figure A.9.3-13). Analysis for the top layer can incorporate load spread effects.



a) Analysis 1 — Layer 2 over layer 3 **b) Analysis 2 — Layer 1 over layers (2 and 3)**

Key

1 layer 1

2 layer 2

3 layer 3

V_L available spudcan reaction see Formula (A.9.3-1)

Figure A.9.3-13 — Spudcan bearing capacity analysis — Three-layer case

A.9.3.3 Yield interaction

A.9.3.3.1 General

During preloading, the soil beneath the spudcan fails plastically and the spudcan penetrates until the bearing capacity is in equilibrium with the preload reaction. When the preload is removed, the soil unloads on the small strain unload-reload stiffness curve. The spudcan geometry and the soil properties at the penetrated position are then used to determine the maximum moment and horizontal capacities that, with the vertical capacity, are the principal values that define the size of the yield interaction surface.

The limiting combinations of the spudcan moment, vertical and horizontal reactions are defined by the yield interaction surface; see Figure A.9.3-14. Inside the yield surface the foundation behaviour is considered to be elastic for small strains, but it becomes increasingly inelastic as the yield surface is approached. On the yield surface, the foundation undergoes inelastic deformation with increased reaction beneath the spudcan. Provided the jack-up's preload capacity is appropriate for a site's environmental conditions, the majority of the foundation load-deflection behaviour during a storm should be essentially elastic and only a few, if any, extreme events cause stiffness reduction.

When the foundation is considered as pinned, the yield surface degenerates to a vertical-horizontal load space.

A.9.3.3.2 to A.9.3.6.7 are generally applicable to spudcan foundation assessment. In some circumstances, the foundation capacities and stiffnesses are not sufficient for the unit to pass the acceptance criteria based on the applied preload. In such cases, the assessment may, where applicable, be based on the foundation capacities and stiffnesses calculated based on soil strength parameters instead of the applied preload in accordance with the approach described in E.4. In such assessments the guidance in these sections should be supplemented by the guidance in E.4. Additional guidance on spudcans fitted with skirts is provided in A.9.4.1.

The modelling approach to the interaction of vertical, horizontal and rotational forces on the spudcan was initially developed for shallow foundations based on a plasticity relationship; see Dean et al. (1995)[56], Cassidy et al. (2006)[47], Wong and Murff (1994)[204], Baerheim (1993)[22] and van Langen and Hospers (1993)[193]. The plasticity relationship can account for moment softening at high loading levels, unloading behaviour and work-hardening effects. The shape of the yield surface for shallow foundations is paraboloidal.

In clay, a deeply embedded spudcan can achieve a greater moment capacity than a spudcan with a shallow penetration [see Templeton et al. (2003)[189], (2005)[190] and Templeton (2006)[185]]. In addition, the shape of the yield surface changes from paraboloidal to becoming progressively more ellipsoidal with increasing penetration. This was first shown experimentally by Martin and Houlsby (2000)[134], further substantiated via numerical analysis by Martin and Houlsby (2001)[135] and confirmed via finite element analysis by Templeton et al. (2005)[189]. This effect can be taken into account by interpolating between the paraboloidal shape of the shallow embedment yield surface [obtained by setting $a = 0$ in Formula (A.9.3-16)] and the ellipsoidal shape for deep embedments ($D_{embed} > 2{,}5B$) using the depth interpolation parameter, a. Accomplishment of the necessary interpolation via a single parameter linear variation of the coefficients was shown to be sufficiently accurate by Templeton (2006)[185].

This model does not include sliding; where sliding is important, this should be incorporated separately using the method described in A.9.3.5.

There is currently no existing data that can be used to justify increases of horizontal and moment capacity, or change of yield surface shape, for deeply embedded spudcans in sand. The application of the yield surface calibrated to shallow penetrations is likely to be conservative for the deep penetration case.

In the yield formula, the gross ultimate vertical bearing capacity, Q_V, is initially established by preload operations and related to v_{Lo} as specified by Formula (A.9.3-6). However, in some cases, subsequent environmental actions can cause further penetration and a corresponding increase in $Q_{V,}$ as is consistent with the load-penetration formulae given in A.9.3.2.2 through A.9.3.2.6. In assessment analyses that incorporate work hardening, such possible increases in Q_V can be included automatically. In other types of analyses, the effects of such increases in Q_V can be included via calculations using the load-penetration formulae, together with values of any additional penetration. In either case, care should be taken to include all contributions from P-Δ effects associated with leaning due to the additional penetration.

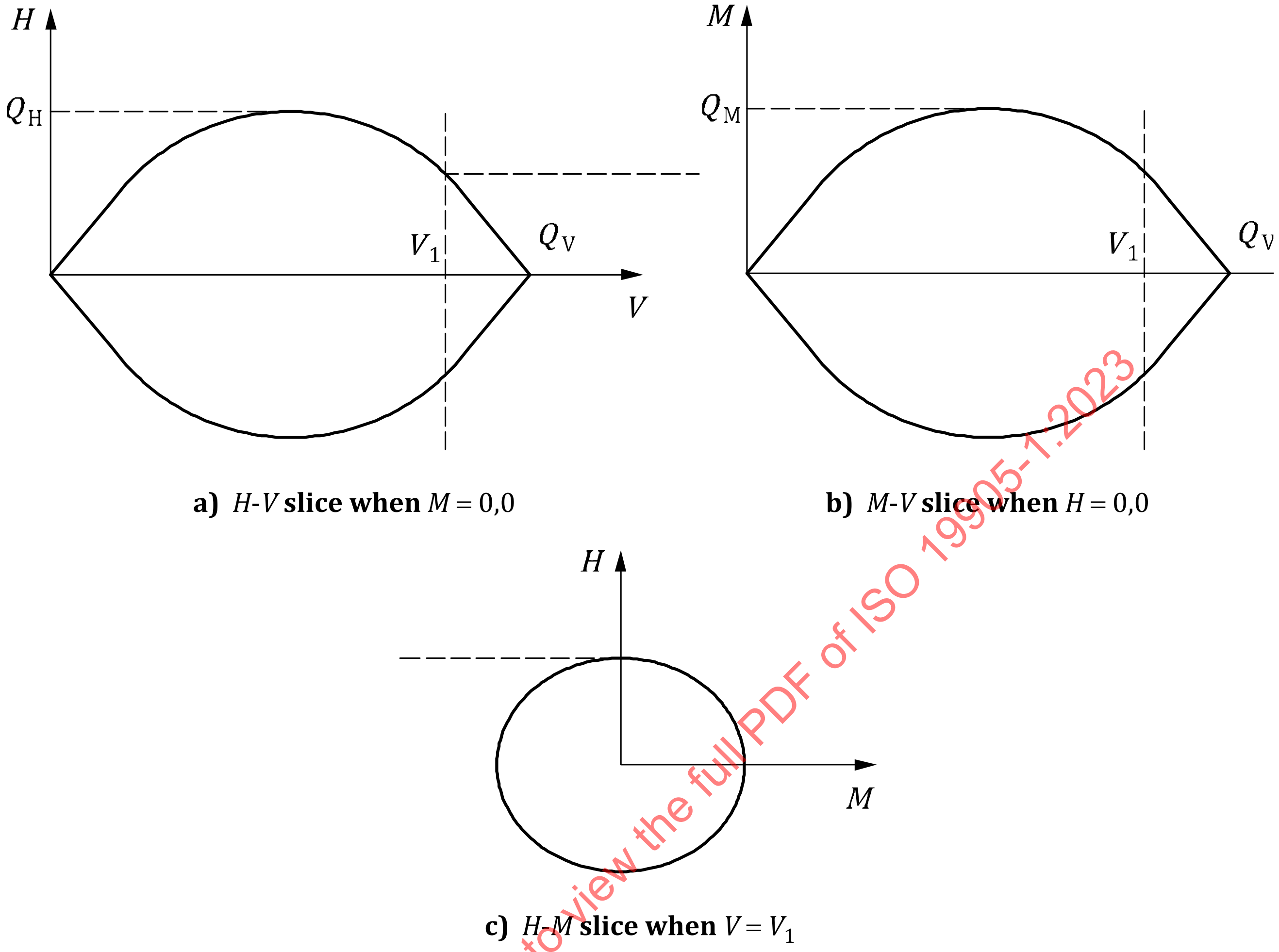
The forces F_H and F_V and the moment F_M acting on the spudcan are the forces transferred to the foundation by the jack-up in operational, extreme storm or earthquake conditions due to the assessment load case F_d in 8.8. They include quasi-static contributions due to factored actions, and contributions from dynamic response, as appropriate, in accordance with the procedures of Clause 10.

— F_H is the horizontal force applied to the spudcan due to the assessment load case F_d (see 8.8).

— F_V is the gross vertical force acting on the soil beneath the spudcan due to the assessment load case F_d (see 8.8).

— F_M is the moment applied to the spudcan due to the assessment load case F_d (see 8.8).

If a force combination (F_V,F_H,F_M) satisfies Formula (A.9.3-16) for the interaction yield surface, then this combination lies on the yield surface. The force combination (F_V,F_H,F_M) lies outside the yield surface if the left-hand side of Formula (A.9.3-16) is greater than zero. Conversely, the force combination lies inside the yield surface if the left-hand side is less than zero.



Key

H	horizontal capacity
M	moment capacity
V	vertical capacity

Figure A.9.3-14 — Three slices through the three-dimensional yield surface (at $M = 0{,}0$; $H = 0{,}0$; and $V = V_1$ constant)

A.9.3.3.2 Ultimate vertical/horizontal/rotational capacity interaction function for spudcans in sand and clay

The general formula, Formula (A.9.3-16), from Templeton (2006)[185] can be used for fully or partially penetrated spudcans:

$$\left[\frac{F_H}{Q_H}\right]^2 + \left[\frac{F_M}{Q_M}\right]^2 - 16(1-a)\left[\frac{F_V}{Q_V}\right]^2\left[1-\frac{F_V}{Q_V}\right]^2 - 4a\left[\frac{F_V}{Q_V}\right]\left[1-\frac{F_V}{Q_V}\right] = 0 \tag{A.9.3-16}$$

where, for the vertical direction:

Q_V is the gross ultimate vertical bearing capacity of the soil beneath the spudcan. Where the spudcans are to be founded in clay soils, this capacity should be calculated considering the effects of cyclic loading, as described in A.9.3.3.7. In the absence of additional penetration, the gross ultimate vertical bearing capacity, Q_V, is equal to the sum of the net vertical bearing capacity achieved during preloading, Q_{Vnet}, multiplied by the cyclic degradation factor, $f_{cy,V}$, and the overburden component, per Formula (A.9.3-18).

Q_H is the ultimate horizontal bearing capacity of the soil beneath the spudcan. Where the spudcans are to be founded in clay soils, this capacity should be calculated incorporating the effects of cyclic loading, as described in A.9.3.3.7;

Q_M is the ultimate moment bearing capacity of the soil beneath the spudcan. Where the spudcans are to be founded in clay soils, this capacity should be calculated incorporating the effects of cyclic loading, as described in A.9.3.3.7;

F_V is the gross vertical force acting on the soil beneath the spudcan due to the assessment load case, F_d (see 8.8) as given in Formula (A.9.3-17):

$$F_V = V_{st} - B_S \quad \text{(with no backfill)}$$

$$F_V = V_{st} + W_{BF,o} + W_{BF,A} - B_S \quad \text{(with backfill)} \quad \text{(A.9.3-17)}$$

V_{st} is the vertical force applied to the spudcan due to the assessment load case, F_d (see 8.8), which includes quasi-static contributions due to factored actions and contributions from dynamic response, as appropriate, in accordance with the procedures of Clause 10, and also includes leg weight and water buoyancy but excludes the submerged weight of backfill ($W_{BF,o} + W_{BF,A}$) and spudcan soil buoyancy (B_S);

where, for the horizontal direction and moment,

F_H is the horizontal force applied to the spudcan due to the assessment load case, F_d (see 8.8);

F_M is the bending moment applied to the spudcan due to the assessment load case, F_d (see 8.8).

a) The clay formulation is given in Formulae (A.9.3-18) to (A.9.3-20) [variables for sand can be found in b)]. The vertical, horizontal and moment capacities Q_V, Q_H and Q_M are calculated in accordance with Formula (A.9.3-18), (A.9.3-19) and (A.9.3-20), respectively.

$$Q_V = (f_{cy,V}\, Q_{Vnet}) + (p'_o\, \pi\, B^2/4) \quad \text{(see A.9.3.3.7)} \quad \text{(A.9.3-18)}$$

$$Q_H = f_{cy,H}\, C_H\, Q_{Vnet} \quad \text{(see NOTE 1 and A.9.3.3.7)} \quad \text{(A.9.3-19)}$$

$$Q_M = f_{cy,M}\, C_M\, Q_{Vnet}\, B \quad \text{(see NOTE 1 and A.9.3.3.7)} \quad \text{(A.9.3-20)}$$

where

$f_{cy,V}$, $f_{cy,H}$ and $f_{cy,M}$ are defined in A.9.3.3.7.

$$Q_{Vnet} = (s_u \cdot N_c \cdot s_c \cdot d_c)\, \pi\, B^2/4 \quad \text{(A.9.3-21)}$$

C_H and C_M are determined distinguishing between clays with i) undrained shear strength linearly increasing with depth (from negligible strength at the mudline), ii) constant undrained shear strength and iii) strength profiles intermediate to those of i) and ii).

i) For clays with undrained shear strength linearly increasing with depth (from negligible strength at the mudline) the formulations are given in Formulae (A.9.3-22) to (A.9.3-23).

The proposed bearing capacity formulae should be used in normally consolidated to lightly overconsolidated clays in which the undrained shear strength increases linearly with depth with a normalised heterogeneity ratio $\rho_{su}B/s_{um}$ that is equal or greater than 1,5.

$$C_H = C_{H,shallow} = 0{,}127 \quad \text{for } D_{embed} < H_{cav}$$
$$= f_{H,deep} \cdot C_{H,deep} \quad \text{for } D_{embed} \geq H_{cav} + 0{,}5B \quad \text{(A.9.3-22)}$$

$$C_M = C_{M,shallow} = 0{,}083 \quad \text{for } D_{embed} < H_{cav}$$
$$= C_{M,deep} \quad \text{for } D_{embed} \geq H_{cav} + 0{,}5B \quad \text{(A.9.3-23)}$$

Values of C_H and C_M for spudcan penetration depths between H_{cav} and $H_{cav} + 0{,}5B$ should be linearly interpolated.

NOTE 1 $C_{H,shallow}$ and $C_{M,shallow}$ are applicable to cases where the cavity above the spudcan remains open (Martin and Houlsby (2001)[135]). $C_{H,deep}$ and $C_{M,deep}$ are based on centrifuge experiments and numerical analyses of deeply penetrated spudcans in normally consolidated clay (Zhang et al. (2013)[218], (2014c)[221]). These are given in Table A.9.3-4 in relation to soil sensitivity, S_t, and spudcan penetration depth, D_{embed}/B. Two-way linear interpolation is suggested to calculate $C_{H,deep}$ and $C_{M,deep}$ for S_t and D_{embed}/B.

The yield surface is paraboloidal, with no evidence of a change of shape to ellipsoidal with increasing penetration depth (Zhang et al. (2014a)[219], (2014b)[220]). Hence, a = 0.

The effect of laterally projected area ratio A_s/A on $C_{H,deep}$ is expressed as factor $f_{H,deep}$ in Figure A.9.3-15. A_S is the spudcan laterally projected embedded area (the projection of the area in contact with the soil) and A is the spudcan effective bearing area based on cross-section taken at uppermost part of bearing area in contact with soil (see Figure A.9.3-2). $C_{M,deep}$ was found to be unaffected by the spudcan aspect ratio. See Zhang et al. (2012b)[220].

Table A.9.3-4 — Yield surface parameters $C_{H,deep}$, $C_{M,deep}$ (after Zhang et al. (2014c)[221]) for $D_{embed} \geq H_{cav} + 0{,}5B$

S_t	D_{embed}/B	$C_{H,deep}$	$C_{M,deep}$
1	0,5	0,186	0,090
	1	0,228	0,095
	1,5	0,277	0,102
	2	0,303	0,106
	3	0,325	0,107
2,2	0,5	0,170	0,089
	1	0,206	0,092
	1,5	0,248	0,098
	2	0,267	0,101
	3	0,286	0,104
3	0,5	0,168	0,088
	1	0,199	0,090
	1,5	0,233	0,096
	2	0,255	0,099
	3	0,278	0,102
4	0,5	0,165	0,087
	1	0,193	0,088
	1,5	0,216	0,093
	2	0,236	0,096
	3	0,257	0,099

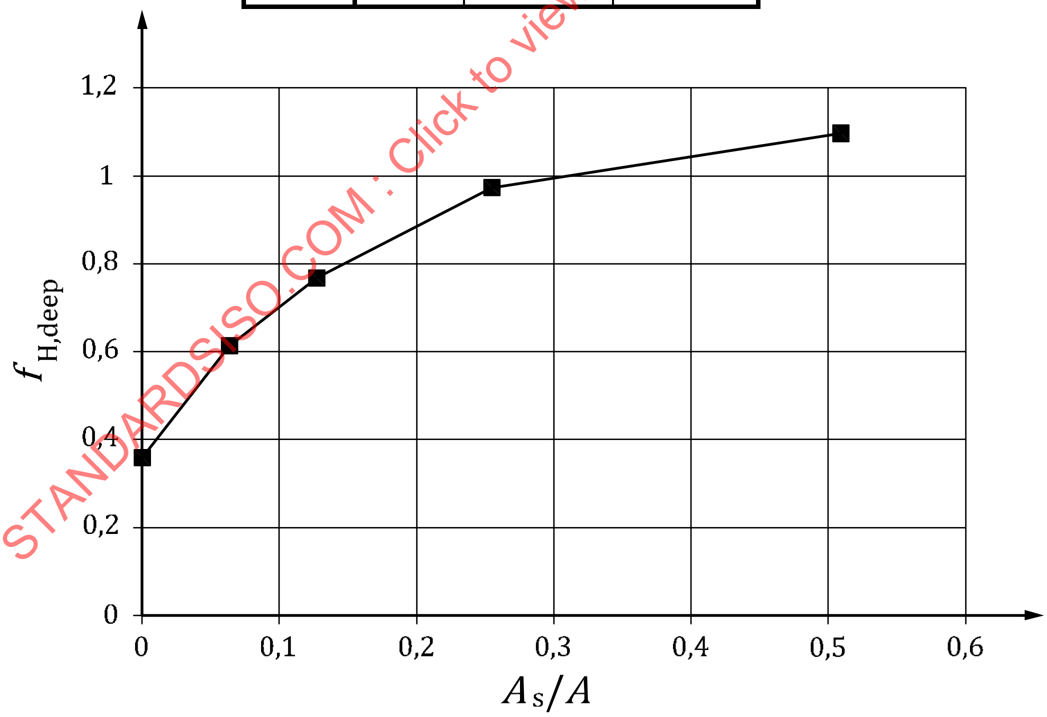


Figure A.9.3-15 — Effect of spudcan aspect ratio A_s/A on $C_{H,deep}$ [after Zhang et al. (2012b)[220]]

ii) For clays with constant undrained shear strength the formulations are given in Formulae (A.9.3-24) to (A.9.3-25):

$$C_H = C_{H,shallow} + (C_{H,deep} - C_{H,shallow})\, D_{embed}/B \quad \text{for } D_{embed} < B \quad \text{(see NOTE 3)} \qquad (A.9.3\text{-}24)$$

$$= C_{Hdeep} \quad \text{for } D_{embed} \geq B$$

$$C_M = [0{,}1 + 0{,}05a(1+b/2)] \qquad (A.9.3\text{-}25)$$

where

$$C_{H,shallow} = [s_{uo}A + (s_{uo} + s_{u,l})\, A_s]/Q_{Vnet} \qquad (A.9.3\text{-}26)$$

$$C_{H,deep} = [1{,}0 + (s_{u,a}/s_{uo})]\,[0{,}11 + 0{,}39(A_s/A)] \quad \text{(with backfill)} \qquad (A.9.3\text{-}27)$$

NOTE 2 The formulation given in Formula (A.9.3-27) for the case of deep embedments in clay is partly based on the finite element results in Templeton (2009)[187], and reduces to Formula 2 in that paper for the case of $s_{u,a} = s_{uo}$,

A is the spudcan effective bearing area based on cross-section taken at uppermost part of bearing area in contact with soil (see Figure A.9.3-2);

A_s is the spudcan laterally projected embedded area (the horizontal projection of the area in contact with the soil),

$$a = D_{embed}/(2{,}5B) \quad \text{for } D_{embed} < 2{,}5B \quad \text{(see NOTE 6)} \qquad (A.9.3\text{-}28)$$

$$= 1{,}0 \quad \text{for } D_{embed} \geq 2{,}5B$$

$$b = (D_b\, s_{u,a})/(D_{embed}\, s_{uo}) \quad \text{(see NOTE 4)} \qquad (A.9.3\text{-}29)$$

D_b is the depth of backflow (see A.9.3.2.1.4), equal to $(D_{embed} - H_{cav})$; infill should not be considered (see NOTE 2);

s_u is the undisturbed undrained shear strength;

$s_{u,a}$ is the undrained shear strength of backfill material above the spudcan, accounting for disturbance and soil sensitivity;

s_{uo} is the undisturbed undrained shear strength at deepest embedment depth of maximum bearing area (D_{embed} below sea floor);

$s_{u,l}$ is the undisturbed undrained shear strength at the spudcan tip.

Formula (A.9.3-27) is only valid for cases including backfill. In cases without backfill $C_{H,deep}$ should be taken as $C_{H,shallow}$ as per Formula (A.9.3-26).

iii) Clays with shear strength profiles intermediate to those of i) and ii).

For clays with undrained shear strength at or near the mudline that is substantial but less than the undrained shear strength near the embedment depth, the following formulae should be used. See Templeton (2021)[188].

For the moment coefficient a linear interpolation should be used:

$$C_M = C_{M,NC} + (s_{u,nml} / s_{u,ned})(C_{M,U} - C_{M,NC}) < C_{M,U} \qquad (A.9.3\text{-}29)$$

where:

C_M is the moment capacity coefficient;

$C_{M,NC}$ is the moment capacity coefficient for the normally consolidated case per A.9.3.3.2 a) i);

$C_{M,U}$ is the moment capacity coefficient for the uniform strength case per A.9.3.3.2 a) ii);

$s_{u\text{-}ned}$ is the minimum undrained shear strength near (within ¼ spudcan diameter below) the embedment depth;

$s_{u\text{-}nml}$ is the minimum (undrained shear) strength within ¼ spudcan diameter below the mudline.

For the horizontal factor a similar linear interpolation should be used:

$$C_H = C_{H,NC} + (s_{u,nml} / s_{u,ned}) (C_{H,U} - C_{H,NC}) < C_{H,U} \quad \text{(A.9.3-30)}$$

where:

C_H is the horizontal capacity coefficient;

$C_{H,C}$ is the horizontal capacity coefficient for the normally consolidated case per A.9.3.3.2 a) i);

$C_{H,U}$ is the horizontal capacity coefficient for the uniform strength case per A.9.3.3.2 a) ii);

$s_{u,ned}$ is the minimum undrained shear strength near (within ¼ spudcan diameter below) the embedment depth;

$s_{u,nml}$ is the minimum undrained shear strength within ¼ spudcan diameter below the mudline.

NOTE 3 For C_H, Templeton (2021)[188] provides a power law interpolation with m = 0,36 for the power law exponent, but with the choice of m=1,0, the power law reduces to the simpler, cautious, linear interpolation of Formula (A.9.3-30).

b) The sand formulation is given in Formulae (A.9.3-31) to (A.9.3-32),

$$Q_H = 0{,}12\left(Q_V - \frac{p'_o \pi B^2}{4}\right) = 0{,}12\, Q_{Vnet} \quad \text{(see NOTE 5)} \quad \text{(A.9.3-31)}$$

$$Q_M = 0{,}075B\left(Q_V - \frac{p'_o \pi B^2}{4}\right) = 0{,}075\, B\, Q_{Vnet} \quad \text{(see NOTE 5)} \quad \text{(A.9.3-32)}$$

$$a = 0{,}0 \quad \text{(see NOTE 6)}$$

where

p'_o is the effective overburden pressure at embedment depth, D_{embed}, of maximum spudcan bearing area;

$Q_{Vnet} = (\gamma' d_\gamma N_\gamma \pi B^3/8) + (p'_o d_q N_q \pi B^2/4) - (p'_o \pi B^2/4)$;

d_γ is the depth factor on self weight for drained soils; $d_\gamma = 1{,}0$;

d_q is the depth factor on surcharge for drained soils,
$d_q = 1 + 2\tan\phi' (1\text{-}\sin\phi')^2 \arctan(D_{embed}/B)$ where $\arctan(D_{embed}/B)$ is in radians;

B is the maximum effective spudcan diameter in contact with the soil;

γ' is the submerged unit weight of the soil;

N_γ and N_q are dimensionless bearing capacity factors calculated for the axisymmetric case (no further shape factor should be applied).

For sand, the values of $0{,}12Q_{Vnet}$ and $0{,}075BQ_{Vnet}$ are based on experimental evidence that includes Tan (1990)[178], Gottardi and Butterfield (1993)[79], (1995)[80], Gottardi et al. (1999)[81], Byrne and Houlsby (2001)[39], Bienen et al. (2006)[24], and Cassidy (2007)[41]. There are no existing data for spudcans deeply embedded in sand. The application of these parameters, which are calibrated to shallow penetrations, is likely to be conservative for the deep penetration case.

At zero vertical loading a shallow sand foundation has no horizontal or moment capacity because it is cohesionless and conforms to the yield interaction formula in bearing. Conversely, for spudcans in clay, when there is adhesion and/or suction, there can be horizontal and moment capacity in excess of the yield interaction surface given above when $F_V < 0{,}5\, Q_V$. In such cases, the yield surface expansion given in A.9.3.3.3 may be used. For deep penetration cases where suction capacity exists, Q_V can be less than zero and the yield surface may be enlarged; the simplified expansion given in A.9.3.3.3 should not be used.

NOTE 4 Both D_{embed} (the depth of embedment) and D_b (the depth of backflow) are measured upward from the lowest elevation of the largest spudcan width. D_b is taken as zero unless the top of the spudcan is effectively covered.

NOTE 5 The horizontal capacity in sand or clay is calculated as a function of the net vertical bearing capacity. The moment capacities are calculated as a function of the product of the net vertical bearing capacity and the effective spudcan diameter. For clay, the net vertical bearing capacity is used because the weight of soil on top of the spudcan does not affect the horizontal and moment capacities. For sand, the use of net capacity is conservative because it neglects the increase in capacity due to the weight of any soil on top of the spudcan which has a beneficial effect on the horizontal and moment capacities. For the case of shallow embedment in clay, a conservative value for C_H can be established by considering minimal embedment of a flat-bottomed spudcan on very strong clay where the horizontal capacity per unit base area is given by the shear strength, and the vertical capacity per unit base area is approximately six times the shear strength, so that: $Q_H = 0{,}16\, Q_{Vnet}$. This value can be used as an alternative, conservative, horizontal capacity expression for shallow embedment in clay.

NOTE 6 The depth interpolation parameter, a, is given as a function of the embedment, D_{embed}, which is measured as the depth below sea floor of the lowest point of the spudcan's maximum width. Technically, $D_{embed} = 0$ does not occur until the spudcan penetration is sufficient to fully seat the spudcan's maximum width. As a practical matter, penetrations shallower than this are not normally expected in clay, but in the event that such shallow penetrations are considered, the value $a = 0$ can be used.

In many cases, simpler forms of the yield interaction formula can be used. Results from finite element analysis [see Templeton et al. (2005)[190] or Templeton (2006)[185]] indicate that insignificant error is incurred by the use of the value, $a = 0$ for embedment less than $0{,}3B$ or by the use of the value, $a = 1$ for embedment greater than $1{,}7B$.

In the case of $a = 0$, the yield interaction formula reduces to the paraboloidal form given in Formula (A.9.3-3):

$$\left(\frac{F_H}{Q_H}\right)^2 + \left(\frac{F_M}{Q_M}\right)^2 - 16\left(\frac{F_V}{Q_V}\right)^2\left(1 - \frac{F_V}{Q_V}\right)^2 = 0 \tag{A.9.3-33}$$

In the case of $a = 1$, the yield interaction formula reduces to the fully ellipsoidal form given in Formula (A.9.3-34):

$$\left(\frac{F_H}{Q_H}\right)^2 + \left(\frac{F_M}{Q_M}\right)^2 - 4\left(\frac{F_V}{Q_V}\right)\left(1 - \frac{F_V}{Q_V}\right) = 0 \tag{A.9.3-34}$$

Formula (A.9.3-16) for the yield surface can be conveniently rewritten to give the maximum available moment on the spudcan F_M as a function of the applied horizontal and vertical forces as given in Formula (A.9.3-35):

$$F_M = Q_M\left[16(1 - a)\left(\frac{F_V}{Q_V}\right)^2\left(1 - \frac{F_V}{Q_V}\right)^2 - \left(\frac{F_H}{Q_H}\right)^2 + 4a\left(\frac{F_V}{Q_V}\right)\left(1 - \frac{F_V}{Q_V}\right)\right]^{0,5} \tag{A.9.3-35}$$

This formula only applies when

$$0 < F_V < Q_V$$

and the condition given in Formula (A.9.3-36) is satisfied:

$$0 < 16(1 - a)\left(\frac{F_V}{Q_V}\right)^2\left(1 - \frac{F_V}{Q_V}\right)^2 - \left(\frac{F_H}{Q_H}\right)^2 + 4a\left(\frac{F_V}{Q_V}\right)\left(1 - \frac{F_V}{Q_V}\right) \tag{A.9.3-36}$$

A.9.3.3.3 Spudcans in clay with $F_V < 0,5\ Q_V$

The yield surface in the region $0 < F_V/Q_V < 0,5$ (typically applicable to windward legs) can be replaced by an adhesion envelope that provides additional horizontal and moment capacity due to spudcan-soil adhesion, The adhesion envelope is applicable for vertical load levels less than $(F_V/Q_V)_t$ which defines the tangent intercept between the adhesion envelope and the standard form of the yield surface and is dependent upon the adhesion factor, α_s , and the "a" parameter that defines the form of the yield surface, The adhesion envelope can be expressed as given in Formula (A.9.3-37):

$$\left(\frac{F_H}{f_1 Q_H}\right)^2 + \left(\frac{F_M}{f_2 Q_M}\right)^2 - 1,0 = 0 \tag{A.9.3-37}$$

where

f_1 is the factor applied to horizontal capacity used in yield surface formula for embedded spudcans on clay, $f_1 = \alpha_s + m_\alpha\left(\frac{F_V}{Q_V}\right)$ (A.9.3-38)

f_2 is the factor applied to moment capacity used in yield surface formula for embedded spudcans on clay, $f_2 = f_1$ where suction (i.e. uplift resistance) is available, or (A.9.3-39)

$$f_2 = \sqrt{16(1-a)\left(\frac{F_V}{Q_V}\right)^2\left(1-\frac{F_V}{Q_V}\right)^2 + 4a\left(\frac{F_V}{Q_V}\right)\left(1-\frac{F_V}{Q_V}\right)}$$ where suction cannot be relied upon;

$\alpha_s = 1{,}0$ for soft clays ($s_u = 20$ to 40 kPa), or (A.9.3-40)

$\alpha_s = 0{,}5$ for stiff clays ($s_u = 75$ kPa to 150 kPa), or

α_s is determined by linear interpolation when $40 < s_u < 75$;

m_α is the gradient of the adhesion envelope,

Figure A.9.3-16 provides a graphical representation of the adhesion envelope and the definitions of the parameters m_α and $(F_V/Q_V)_t$,

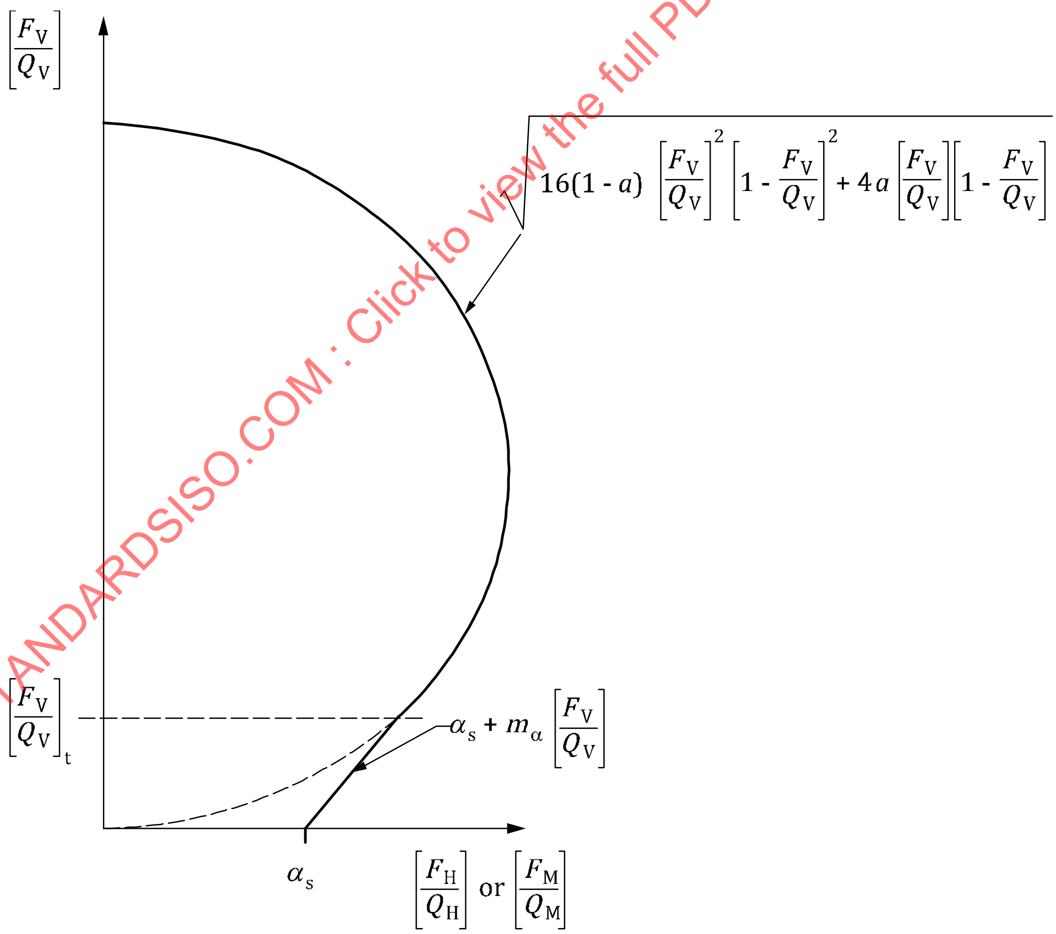


Figure A.9.3-16 — Illustration of the adhesion envelope modification to the standard yield surface for $F_V < \left(\frac{F_V}{Q_V}\right)_t$

α_s is the adhesion factor and accounts for the degree of adhesion. The assessor should consider α_s values within the range of 0,5 to 1,0 depending on site-specific soil data, spudcan/soil interface roughness, etc. When hard clay is present at the surface with an α_s value below 0,5, the standard form of the yield surface should be used, see Formula (A.9.3-16).

The values for m_α and $(F_V/Q_V)_t$ have been determined for $a = 0{,}0$ (paraboloidal) as given in Formulae (A.9.3-41) and (A.9.3-42) and for $a = 1{,}0$ (ellipsoidal) as given in Formulae (A.9.3-43) and (A.9.3-44):

— For $a = 0$:

$$m_\alpha = 4\left(1 - \sqrt{\alpha_s}\right) \tag{A.9.3-41}$$

$$\left(\frac{F_V}{Q_V}\right)_t = \sqrt{\frac{\alpha_s}{4}} \tag{A.9.3-42}$$

— For $a = 1$:

$$m_\alpha = \frac{1-\alpha_s^2}{\alpha_s} \tag{A.9.3-43}$$

$$\left(\frac{F_V}{Q_V}\right)_t = \frac{\alpha_s^2}{\alpha_s^2+1} \tag{A.9.3-44}$$

Values of m_α and $(F_V/Q_V)_t$ for intermediate values of a can be solved for iteratively.

Selected values of $(F_V/Q_V)_t$ are provided in Table A.9.3-4:

Table A.9.3-4 — Values of $(F_V/Q_V)_t$ for various values of a and α_s

	a						
α_s	**0,0**	**0,2**	**0,4**	**0,5**	**0,6**	**0,8**	**1,0**
0,5	0,354	0,334	0,308	0,293	0,276	0,238	0,200
0,6	0,387	0,373	0,354	0,343	0,331	0,300	0,265
0,7	0,418	0,408	0,396	0,388	0,379	0,357	0,329
0,8	0,447	0,441	0,433	0,428	0,423	0,409	0,390
0,9	0,474	0,471	0,468	0,465	0,463	0,457	0,448
1,0	0,500	0,500	0,500	0,500	0,500	0,500	0,500

Selected values of m_α are provided in Table A.9.3-5:

Table A.9.3-5 — Values of m_α for various values of a and α_s

	a						
α_s	**0,0**	**0,2**	**0,4**	**0,5**	**0,6**	**0,8**	**1,0**
0,5	1,172	1,200	1,239	1,264	1,295	1,378	1,500
0,6	0,902	0,917	0,937	0,950	0,965	1,006	1,067
0,7	0,653	0,661	0,670	0,676	0,683	0,701	0,729
0,8	0,422	0,425	0,429	0,431	0,434	0,440	0,450
0,9	0,205	0,206	0,207	0,207	0,208	0,209	0,211
1,0	0,000	0,000	0,000	0,000	0,000	0,000	0,000

Formula (A.9.3-35) can be re-written to give the maximum moment on the spudcan as a function of the horizontal force as given in Formula (A.9.3-45):

$$F_M = f_2 Q_M \left[1 - \left(\frac{F_H}{f_1 Q_H} \right)^2 \right]^{0,5} \tag{A.9.3-45}$$

Formula (A.9-45) applies only when the conditions given in Formulae (A.9.3-46) and (A.9.3-47) are satisfied:

$$0 < \frac{F_V}{Q_V} < \left(\frac{F_V}{Q_V} \right)_t \tag{A.9.3-46}$$

and

$$F_H < f_1 Q_H \tag{A.9.3-47}$$

For a vertical and horizontal force combination that lies inside the yield surface given in A.9.3-45, the moment on the spudcan is limited to the maximum available moment capacity Q_M.

A.9.3.3.4 Modification of the yield surface for partial penetration in sand

On seabeds of silica sands, conical spudcans that are not fully seated can develop increased moment capacity due to the rotation of the spudcan causing an eccentric seabed reaction which provides a beneficial resisting moment.

The effect may be taken into account for spudcans with $F_V/Q_V > 0,5$. The increased ultimate moment capacity Q_{Mp} due to eccentric seabed reaction is estimated as the minimum of Q_{Mps} and Q_{Mpv}, calculated from Formulae (A.9.3-48) and (A.9.3-49) respectively; see Svanø (1996)[175]:

$$Q_{Mps} = 0,075 B Q_{Vnet} (B_{max} / B)^3 \tag{A.9.3-48}$$

$$Q_{Mpv} = 0,15 B F_V \tag{A.9.3-49}$$

Note that the horizontal capacity is unaffected.

The combined capacity should be checked against the modified yield interaction surface given in Formula (A.9.3-50):

$$\left(\frac{F_H}{Q_H} \right)^2 + \left(\frac{F_M}{Q_{Mp}} \right)^2 - 16 \left(\frac{F_V}{Q_V} \right)^2 \left(1 - \frac{F_V}{Q_V} \right)^2 = 0 \tag{A.9.3-50}$$

A.9.3.3.5 Expansion of the yield surface for additional penetration in sand

Additional penetration of a spudcan in sands can be accounted for by using plasticity principles. Recommendations on updating stiffness and the flow of plastic displacements within a work-hardening framework are provided in Houlsby and Cassidy (2002)[98], Cassidy et al. (2002a)[43] and Bienen et al. (2006)[24].

This increase in penetration can also result in increased structural utilizations, which should be assessed; see A.9.3.6.6.

A.9.3.3.6 Expansion of the yield surface for additional penetration in clay

For additional penetration of spudcans in clay, Wong and Murff (1994)[204] and van Langen and Hospers (1993)[193] provide work-hardening modifications to the yield surface formulae. Updated stiffnesses and capacities are determined through plasticity principles.

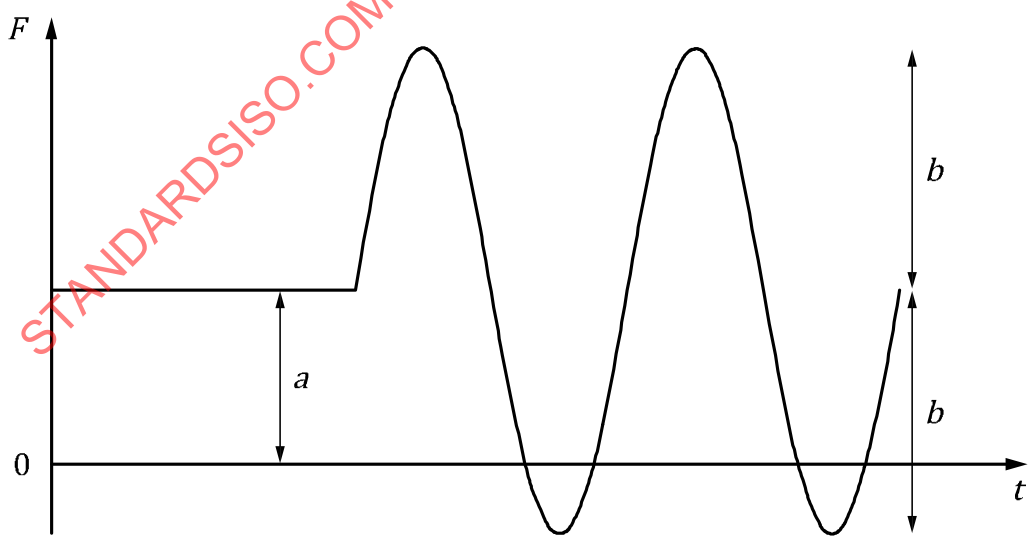
A.9.3.3.7 Effect of cyclic loading on the yield surface

A.9.3.3.7.1 General

According to Andersen (2015)[20] and NGI (2018)[141], the degradation of clays under cyclic loading can reduce the capacity of offshore foundations. As the foundations of jack-up units are subjected to cyclic loading during storm events, this soil behaviour should be considered in a site assessment. The results of laboratory tests of soil samples under cyclic loading indicate that the cyclic undrained shear strength of clays under pure two-way cyclic loading (i.e., where the average load is zero) can be less than the corresponding static undrained shear strength. The magnitude of this reduction is dependent on the soil's plasticity index, I_p, and has some dependency on the overconsolidation ratio, R_{OC}, as indicated in Figure A.9.3-18, adapted from Andersen (2015)[20].

When there is a combination of cyclic and non-zero average loading, the soil strength can be considered to comprise of average and cyclic strength components. Depending on the ratio of the cyclic to average loading during the wave event being assessed, this soil strength can be less than, equal to or greater than the static soil strength. The guidance below, taken from NGI (2018)[141], provides a generalized and simplified approach to estimate the effect of cyclic load on the foundation capacity in clay under combined average and cyclic load components on a leg in the absence of appropriate site-specific cyclic soil strength data, see Figure A.9.3-17.

The definition of the average (a) and the cyclic (b) load components is presented in Figure A.9.3-17.



Key

a average loading component
b cyclic loading component
F footing loading (on one leg)
t time

Figure A.9.3-17 — Average and cyclic loading components as per NGI (2018)[141]

The ultimate bearing capacity of a foundation in clay can be determined from Formulae (A.9.3-18) to (A.9.3-20) by multiplying each of the static vertical, horizontal and moment bearing capacities by a corresponding cyclic degradation factor, $f_{cy,V}$, $f_{cy,H}$ and $f_{cy,M}$ respectively. This procedure should be performed for each spudcan separately as $F_{a,i}$ and/or Q_i can vary between spudcans. Due to the simplified nature of the formulation, the ultimate bearing capacity obtained in this manner is limited such that it does not exceed the static ultimate bearing capacity.

Guidance on implementing cyclic degradation effects in clay for assessments based on calculated foundation capacities is given in E.4.

The unfactored average individual force component on the foundation, $F_{a,i}$, is defined as the average of the peak and trough of the total foundation loading for the assessment load case (i.e. including all force contributions due to co-linear actions from wind, wave, current, inertia, weight and buoyancy, etc. during one wave cycle) where the subscript, *i*, refers to each of the vertical, horizontal and moment average force components on each foundation. It is, in most cases, conservative to calculate the average forces by considering only the unfactored co-linear wind and current actions for each storm direction, as specified in the assessment load case.

The average vertical force acting on each spudcan depends upon the storm heading direction, e.g. whether each spudcan is the windward or leeward leg; it should be expressed in terms of net vertical force, i.e. the gross vertical force, F_V, less the overburden weight, $\left(p_o^{'}\pi B^2\right)/4$

The cyclic degradation factor, $f_{cy,i}$ can be determined for each load component *i* (vertical, horizontal, moment) as given in Formula (A.9.3-51) and should not exceed 1,0:

$$f_{cy,i} = f_{cy,0} + F_{a,i}/Q_i \leq 1{,}0 \qquad \text{(A.9.3-51)}$$

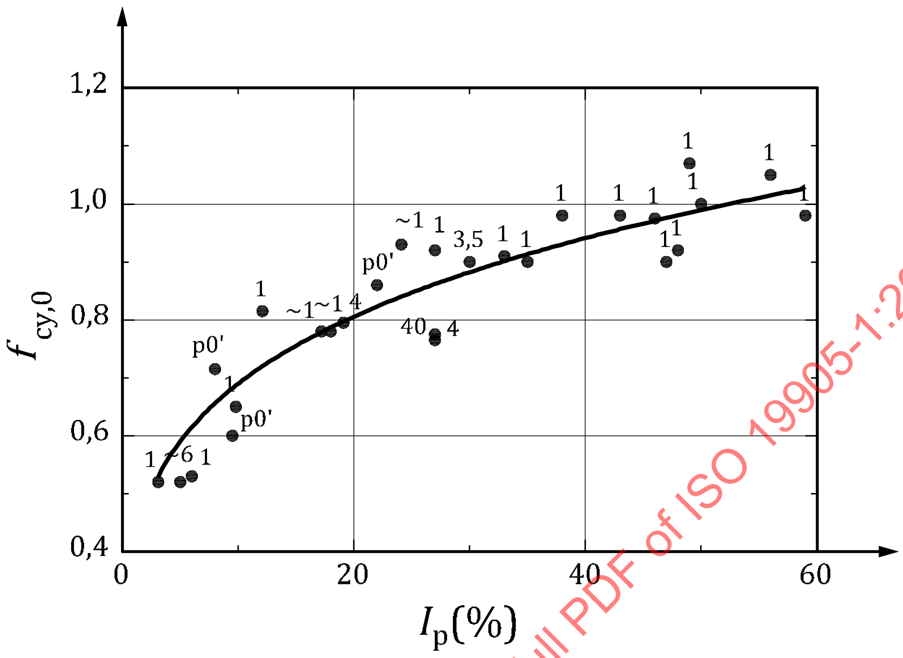
where

$f_{cy,0}$ is an inherent soil property that represents the ratio of the undrained shear strength ($\tau_{cy,f}$) under pure two-way cyclic loading (i.e. zero average load) to the static undrained shear strength (s_u). These recommendations, and the data plotted in Figure A.9.3-18, are valid for non-calcareous and non-cemented clays only. The majority of the data presented is for normally consolidated soils. The few data points for $R_{OC} > 1$ soils do not seem to indicate a strong effect of R_{OC}. However, there are not sufficient data to draw strong conclusions with respect to the effect of R_{OC} upon cyclic degradation.

$F_{a,i}$ is the unfactored average vertical, horizontal or moment force component over a wave cycle on the foundation of one leg (see above). $F_{a,v}$ is the average component of the unfactored net vertical footing force given by $F_V - \left(p_o^{'}\pi B^2\right)/4$, where F_V is defined in Formula (A.9.3-17).

Q_i is the corresponding unfactored static vertical (Q_{Vnet}), horizontal or moment bearing capacity (i.e. without cyclic degradation), see A.9.3.3.7.2.

A detailed evaluation of the average and cyclic load components at each spudcan for each assessment load case can be substituted by a simplified conservative calculation of $f_{cy,i}$. Guidance on this simplified approach is provided in A.9.3.3.7.3



Key

Data Labels	Overconsolidation ratio, R_{OC}. Points labelled p0’ refer to tests on normally consolidated clays that have been consolidated to the in situ effective stress, p0’. ~ means approximately.
Line of best fit	$0{,}41\ I_p^{0{,}224}$
I_p	plasticity index in percent
$f_{cy,0}$	ratio of the cyclic to static shear strength of the soil.

Figure A.9.3-18 — Ratio between cyclic and static undrained shear strengths of non-calcareous and non-cemented clays for ten equivalent cycles as a function of plasticity index (valid for symmetrical direct simple shear cyclic loading) [Adapted from Figure 12.31 in Andersen (2015)[20]]

A.9.3.3.7.2 Assumptions

The methodology described in A.9.3.3.7.1 is based on the following assumptions:

— Static foundation capacities (Q_i) are calculated using undrained shear strengths, that are applicable for the vertical, horizontal and moment failure modes within the soil, and are obtained using a standard laboratory testing rate of shear strain of 3 to 5% per hour;

— That calculations indicating that normalised cyclic shear strengths can be used to estimate normalized cyclic capacities (NGI, 1992)[140];

— That a unidirectional cyclic loading history that can be represented by 10 equivalent cycles of the maximum load in the storm with a load period of 10 seconds. This can be conservative for normally and lightly overconsolidated clays (NGI, 1992)[140]. The approach described in A.9.3.3.7 is formulated considering extreme storm conditions and is not valid for cyclic loading due to earthquakes.

— The cyclic degradation parameters being applied to the net foundation bearing capacities and $f_{cy,i}$ being evaluated using the ratio of the average net vertical, horizontal or moment force component on the foundation to the net static vertical, horizontal or moment foundation capacity.

A.9.3.3.7.3 Simplified conservative calculation of $F_{a,i}$

The following simplified conservative assessment approach can be used to evaluate the cyclic degradation parameters, $f_{cy,i}$.

— If $F_{a,V}/Q_{Vnet} > 0,3$ then $f_{cy,V} = 1,0$ where $F_{a,V}$ can be taken as the lowest value of $F_V - p'_o \pi B^2/4$ for all legs under still water conditions;
else
$f_{cy,V} = f_{cy,0} + F_{a,V}/Q_{Vnet} \leq 1,0$

— $f_{cy,H} = f_{cy,0} + F_{a,H}/Q_H \leq 1,0$ where $F_{a,H}$ can, in this simplified approach, be taken as the total unfactored average horizontal load on the jack-up due to horizontal co-linear wind and current actions divided by the number of legs;

— $f_{cy,M} = f_{cy,0}$ based on the conservative assumption that $F_{a,M} = 0,0$.

Alternatively, in the absence of information on the average force components, $f_{cy,H} = f_{cy,M} = f_{cy,0}$ for the horizontal and moment capacities and $f_{cy,V}$ can be taken as 1,0 if $F_{a,V}/Q_{Vnet} > 0,3$.

- NOTE 1 $F_{a,V}$ will typically exceed 30% of the net vertical static bearing capacity, resulting in a vertical cyclic degradation factor, $f_{cy,V}$, of 1,0 in most cases. The horizontal and moment force components can be more symmetrical, i.e. have relatively small average force components; consequently, the assumption that these average load components are zero will result in cyclic degradation factors $f_{cy,H} = f_{cy,M} = f_{cy,0}$.
- NOTE 2 Although the ratio $F_{a,V}/Q_{Vnet}$ can be less than 0,3 for windward legs for the assessment load case, for such vertical load levels the horizontal foundation capacity, Q_H, is mostly governed by the soil strength parameters, rather than the vertical foundation capacity. Consequently, implementation of cyclic degradation of the windward legs' vertical capacity for such load cases is not necessary as it would not reduce the horizontal and moment capacities, see A.9.3.3.3.

A.9.3.4 Foundation stiffness

A.9.3.4.1 Vertical, horizontal and rotational stiffness

Vertical and horizontal stiffnesses of the foundation are based on the elastic solutions for a rough flat-based circular rigid disk on an elastic half-space with modification factors to account for spudcan embedment. For the effects of leg embedment, see A.9.3.4.6. The elastic stiffness factors are calculated assuming full contact of the spudcan with the seabed. If the vertical reaction is insufficient to maintain full contact as the moment increases, then reduced stiffnesses should be used. The stiffness factors are derived for a homogeneous, linear, isotropic soil as given in Formulae (A.9.3-52) to (A.9.3-54):

$$K_1 = K_{d1} \frac{2G_{max}B}{(1-\nu)} \quad \text{(vertical stiffness)} \qquad \text{(A.9.3-52)}$$

$$K_2 = K_{d2} \frac{16G_{max}B(1-\nu)}{(7-8\nu)} \quad \text{(horizontal stiffness)} \qquad \text{(A.9.3-53)}$$

$$K_3 = K_{d3} \frac{G_{max}B^3}{3(1-\nu)} \quad \text{(rotational stiffness for relatively low levels of loading; see Winterkorn and Fang (1975)[202])} \qquad \text{(A.9.3-54)}$$

Torsional spudcan foundation stiffness (i.e. for spudcan rotation about its vertical axis) should not be used.

The selection of the small-strain shear modulus of the foundation soil, G_{max}, is discussed in A.9.3.4.3 to A.9.3.4.5. A high or a low representative value should be selected as appropriate for the analysis being undertaken, e.g. the upper value is appropriate for fatigue related analysis. The shear modulus is influenced by the stress level and strain amplitude. In general, the shear modulus decreases with increasing strain amplitude. In this document, the consequences are addressed by reducing the stiffnesses (see A.9.3.4.2.2).

NOTE Although the cross-coupling stiffness, K_4, which links horizontal footing displacements and footing rotations to moment and horizontal loads, respectively, is not explicitly calculated, it is incorporated to some extent by the choice of the seabed reaction point as described in A.8.6.2.

A.9.3.4.2 Stiffness modifications

A.9.3.4.2.1 Embedment

Table A.9.3-6 provides values for the stiffness depth factors K_{d1}, K_{d2} and K_{d3}, to account for embedment effects on the stiffness of flat plate and conical type footings on an elastic half space, after Bell (1991)[23]. Values for the case of partial backfill can be interpolated from the values for full and no backfill provided in the tables. Zhang et al. (2012a)[216] also present stiffness depth factors for typical spudcan-shaped footings in undrained clay (Poisson's ratio, ν = 0,5) based on a constant rigidity index profile with depth; care is required when using this approach for soil profiles where this is not the case, e.g. where the overconsolidation ratio is not constant with depth.

For embedment depths $2D/B$ greater than 4,0, the stiffness depth factors for $2D/B$ = 4,0 should be used (data extrapolation is not recommended).

Table A.9.3-6 — Stiffness depth factors

Stiffness factors for $\nu = 0,0$						
$2D/B$	K_{d1}		K_{d2}		K_{d3}	
	No backfill	**Full backfill**	**No backfill**	**Full backfill**	**No backfill**	**Full backfill**
0,0	1,00	1,00	1,00	1,00	1,00	1,00
0,5	1,15	1,21	1,33	1,49	1,28	1,64
1,0	1,28	1,41	1,44	1,71	1,43	2,05
2,0	1,42	1,70	1,51	1,92	1,51	2,31
4,0	1,59	2,00	1,61	2,06	1,57	2,41

Stiffness factors for $\nu = 0,2$						
$2D/B$	K_{d1}		K_{d2}		K_{d3}	
	No backfill	**Full backfill**	**No backfill**	**Full backfill**	**No backfill**	**Full backfill**
0,0	1,00	1,00	1,00	1,00	1,00	1,00
0,5	1,11	1,18	1,32	1,47	1,23	1,54
1,0	1,21	1,34	1,42	1,67	1,37	1,90
2,0	1,34	1,59	1,48	1,85	1,44	2,15
4,0	1,49	1,85	1,58	1,98	1,51	2,25

Stiffness factors for $\nu = 0,4$						
$2D/B$	K_{d1}		K_{d2}		K_{d3}	
	No backfill	**Full backfill**	**No backfill**	**Full backfill**	**No backfill**	**Full backfill**
0,0	1,00	1,00	1,00	1,00	1,00	1,00
0,5	1,08	1,14	1,31	1,45	1,18	1,43
1,0	1,16	1,27	1,41	1,64	1,31	1,76
2,0	1,27	1,48	1,48	1,80	1,39	2,01
4,0	1,41	1,72	1,57	1,92	1,47	2,13

Stiffness factors for $\nu = 0,5$						
$2D/B$	K_{d1}		K_{d2}		K_{d3}	
	No backfill	**Full backfill**	**No backfill**	**Full backfill**	**No backfill**	**Full backfill**
0,0	1,00	1,00	1,00	1,00	1,00	1,00
0,5	1,07	1,10	1,32	1,44	1,18	1,39
1,0	1,15	1,23	1,44	1,62	1,31	1,71
2,0	1,25	1,44	1,51	1,78	1,40	1,99
4,0	1,40	1,69	1,59	1,91	1,51	2,16

A.9.3.4.2.2 Linear vertical, linear horizontal and secant rotational stiffness

Except for simple dynamic analyses with linearized foundations contained within A.10.4.4.1.2 Option 1, the following procedure should be used if the reduction of rotational stiffness is not included in the soil model. The method accommodates stiffness reduction in a simple manner for responses within the yield surface.

If the force combination (F_V, F_H, F_M) lies outside the yield surface, the linearized rotational stiffness at the spudcan should be reduced using iterative analysis until the force combination lies on the yield surface.

Although the force combination (F_V, F_H, F_M) lies inside the yield surface, the initial estimate of linearized rotational stiffness should also be reduced by following the iterative procedure in A.10.4.4.1.2 and using the foundation rotational stiffness reduction factor, f_r, which has an increasing effect as the yield surface is approached. The factor is obtained from Formula (A.9.3-55); see Templeton (2007)[186]:

$$f_r = (1 - n)\, r_f / \ln[(1 - n r_f)/(1 - r_f)] \tag{A.9.3-55}$$

The parameter, n, accommodates spudcan rotation resistance curves with various degrees of curvature change. In practice, the value of this parameter should be set to suit the data (either empirical or analytical) applicable to the jack-up and site. Finite element analysis for the Gulf of Mexico (Templeton, 2007)[186] clay types indicate the range of $n = -0{,}25$ to $-1{,}0$, with $n = -0{,}5$ providing the best overall representation. In the absence of directly applicable data, the value of n can be set to 0. In this case, the rotational stiffness reduction factor expression takes the simpler form given in Formula (A.9.3-56):

$$f_r = -r_f / \ln(1 - r_f) \tag{A.9.3-56}$$

As n approaches 1,0 the stiffness reduction expression tends towards the form given in Formula (A.9.3-57), which gives the most conservative treatment of stiffness reduction:

$$f_r = 1 - r_f \tag{A.9.3-57}$$

The variable, r_f, in the stiffness reduction expression is the failure ratio defined by Formula (A.9.3-58):

$$r_f = \frac{\left[\left(\frac{F_H}{Q_H}\right)^2 + \left(\frac{F_M}{Q_M}\right)^2\right]^{0,5}}{\left[16(1-a)\left(\frac{F_V}{Q_V}\right)^2\left(1-\frac{F_V}{Q_V}\right)^2 + 4a\left(\frac{F_V}{Q_V}\right)\left(1-\frac{F_V}{Q_V}\right)\right]^{0,5}} \le 1{,}0 \tag{A.9.3-58}$$

where "a" is as defined in A.9.3.3.2.

NOTE $r_f > 1{,}0$ implies that the force combination (F_V, F_H, F_M) lies outside the yield surface. Under such conditions, the reduced stiffness factor is not applicable, and the rotational stiffness is reduced until the force combination lies on the yield surface.

For fully embedded foundations in clays at vertical force ratio $F_V / Q_V < \left(\frac{F_V}{Q_V}\right)_t$, the failure ratio can be expressed as given in Formula (A.9.3-59):

$$r_f = \left[\left(\frac{F_H}{f_1 Q_H}\right)^2 + \left(\frac{F_M}{f_2 Q_M}\right)^2\right]^{0,5} \leq 1,0 \quad \text{(A.9.3-59)}$$

where $\left(\frac{F_V}{Q_V}\right)_t$, f_1 and f_2 are as defined in A.9.3.3.3.

A.9.3.4.2.3 Non-linear vertical, horizontal and rotational stiffness

A full yield interaction surface model that includes non-linear vertical, horizontal and rotational stiffnesses implicitly incorporates the necessary stiffness reduction as a consequence of work-hardening plastic displacement and rotation [van Langen et al. (1997)[194], Wong et al. (1993)[205], and Cassidy et al. (2004b)[46]]. The stiffness reduction factor should not be applied.

A.9.3.4.2.4 Non-linear continuum foundation model

A continuum foundation model that includes non-linear soil behaviour (e.g. elastic-plastic work hardening) implicitly incorporates the necessary stiffness reduction. The stiffness reduction factor should not be applied.

A non-linear continuum foundation model should not be used unless one of the simpler analysis methods has been used to provide a benchmark for the results.

A.9.3.4.3 Selection of shear modulus, G_{max}, for clay

The value of the small-strain shear modulus for clay, G_{max}, should be based on the value of the undrained shear strength, s_u, measured at the depth $z = D_{embed} + 0,15B$, where B is the effective diameter of the spudcan in contact with the soil and D_{embed} is the predicted embedment depth below the sea floor of the lowest point on the spudcan with diameter B. Where the clay is significantly layered, the average strength within the range $z = D_{embed}$ to $z = D_{embed} + 0,3B$ should be used. Except in areas with carbonate clays or clayey silts the shear modulus should be calculated from Formula (A.9.3-60), see Cassidy et al. (2002b)[44] and Noble Denton (2006)[142]:

$$G_{max} = s_u \frac{600}{(R_{OC})^{0,25}} \quad \text{subject to the limitations given below.} \quad \text{(A.9.3-60)}$$

where

G_{max} is the maximum value of the shear modulus (of the foundation soil), which occurs at small strain;

NOTE In forming estimates of foundation stiffness from linear elastic solutions to represent non-linear soil behaviour, one general method uses the linear elastic stiffness solution with a shear modulus taken as a function of strain level. Another method uses a non-linear stiffness function, which varies with the amplitude of the action and a constant shear modulus. In the former method a distinction is made between G_{max} (the maximum value of the shear modulus, which occurs at small strain) and G (the general shear modulus, which varies with strain magnitude). In the latter method, the maximum value of shear modulus is used and no such distinction in terms is made. The process outlined in A.9.3.4.2.2 adopts the latter approach and hence G_{max} is referred to throughout this document.

R_{OC} is the overconsolidation ratio;

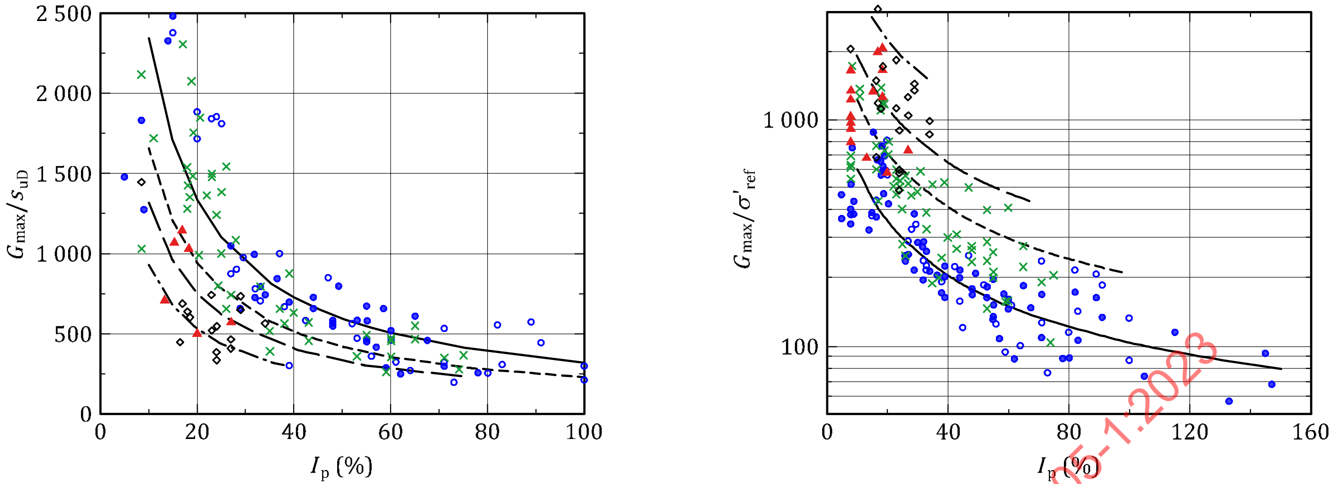
For extreme loading situations, and in the absence of other data, G_{max}/S_u should be conservatively limited to 400; see Noble Denton (2006)[142] which is based on overconsolidated clays with plasticity indices of up to 60 %. Alternatively due consideration should be given to the possibility of determining site-specific shear moduli for cohesive soils other than overconsolidated clays and/or where the plasticity indices exceed 60 %.

$G_{max}/S_u = 600$ is supported by field data for jack-up responses in low R_{OC} clays in the Gulf of Mexico; see Templeton (2006)[185].

In some cases, higher ratios of G_{max}/S_u have been reported and it should be recognized that G_{max}/S_u generally decreases with increasing plasticity index and increases with increasing overconsolidation ratio, as shown in Figure A.9.3-19, reproduced from Figures 11.5 of Andersen (2015)[20]. The normalization with respect to undrained shear strength shown in Figure A.9.3-19a is with reference to the undrained shear strength from direct simple shear tests. Where the shear modulus is not supported by site-specific data, the assessor should account for this trend when determining G_{max}.

The recommendations given above (Cassidy et al., 2002b)[44] are intended for use in site-specific assessments for both extreme loading and applications involving small strain beneath the spudcan.

In the calculation of fixity for extreme loading, the rotational stiffness based on the small-strain G_{max} values is degraded, either explicitly in the linearized foundation model using the stiffness reduction formulae given in A.9.3.4.2.2, or implicitly using non-linear foundation models. In the case of small-strain applications such as in structural fatigue analysis, the stiffness reductions do not apply, and it can be appropriate to adopt a high representative value of G_{max}.



a) G_{max} normalized with respect to undrained shear strength from direct simple shear tests

b) G_{max} normalized with respect to reference effective stress

Key

●	Data – OCR = 1,0
○	Data – OCR = 1,0 – 1,5
×	Data – OCR = 1,5 – 4,0
▲	Data – OCR = 4,0 – 10,0
◇	Data – OCR = 10,0 – 40,0
———	Fit: OCR = 1
– – – –	Fit: OCR = 4
—— ——	Fit: OCR = 10
—·—·—	Fit: OCR = 40
I_p	plasticity index in percent
R_{oc}	over-consolidation ratio
G_{max}	maximum shear modulus that occurs at small strain
s_{uD}	undrained shear strength from direct simple shear tests
σ'_{ref}	reference effective stress defined as $\sigma'_{ref} = p_a(\sigma'_{vc}/p_a)^{0.9}$ where p_a = 100kPa
σ'_{vc}	vertical effective consolidation stress

NOTE 1 The determination of G_{max} via the use of rigidity index is inherently approximate.

NOTE 2 Adapted from Andersen (2015)[20], Figure 11.5.

Figure A.9.3-19 — Normalized small-strain shear modulus as a function of plasticity index and overconsolidation ratio

A.9.3.4.4 Selection of shear modulus, G_{max}, for sand

For sands, the small-strain shear modulus should be computed from Formula (A.9.3-61):

$$\frac{G_{max}}{p_a} = j\left(\frac{V_{sw}}{Ap_a}\right)^{0,5} \tag{A.9.3-61}$$

where

j is the dimensionless stiffness factor, $j = 230\left(0,9 + \frac{D_R}{500}\right)$;

p_a is the atmospheric pressure, typically taken as 101,3 kPa;

D_R is the relative density (expressed in percent);

V_{sw} is the gross vertical spudcan reaction inclusive of backfill under still water conditions (the reaction that would be obtained if the jack-up were supported on an infinitely rigid foundation, plus the reaction due to the submerged weight of any backfill on the spudcan, less the submerged weight of soil displaced by the spudcan below D_{embed}, the greatest embedment depth of maximum cross-sectional spudcan bearing area below the sea floor).

The recommendations given above (Noble Denton Europe and Oxford University, 2006[142] developed from the work of Cassidy et al., 2002b[44] and Wroth et al., 1979[206]) are intended for use in site-specific assessments for both extreme loading and applications involving small strain beneath the spudcan. In the calculation of fixity for extreme loading, the rotational stiffness based on the small-strain G_{max} values is degraded, either explicitly in the linearized foundation model using the stiffness reduction formulae given in A.9.3.4.2.2, or implicitly using non-linear foundation models. In the case of small-strain applications such as in structural fatigue analysis, the stiffness reductions do not apply, and it can be appropriate to adopt a high representative value of G_{max}.

A.9.3.4.5 Selection of shear modulus for layered soils

Roesset (1980)[157] provides formulae for the vertical, horizontal, rotational and torsion stiffnesses of a rigid disc on a layer of finite thickness, including the effect of embedment into that layer. Guidance on soil moduli of multilayered systems is available in Ueshita and Meyerhof (1967)[191].

A.9.3.4.6 Soil-leg interaction

For deep penetrations, typically experienced in soft clay conditions, the calculation of foundation fixity can be augmented with the inclusion of the lateral soil resistance on the leg members (Brekke et al., 1989)[34].

The lateral soil resistance of the backfill material can be modelled based on concepts proposed by Matlock (1970)[136] for lateral soil resistance of piles. The jack-up leg can be modelled as an equivalent pile for purposes of determining *p-y*, or load-deflection curves.

The diameters of the individual members (i.e. leg chords and braces) give appropriate characteristic dimensions for determining the *p-y* curves. The *p-y* curves for each member are directionally combined to form equivalent *p-y* curves along the leg, accounting for soil layering and changes in leg geometry. Any external face of each leg in compressive contact with the soil may be assumed to contribute to the lateral resistance. Typically, equivalent springs at each bay elevation are used to simplify the calculations.

A.9.3.5 Vertical-horizontal foundation capacity envelopes

A.9.3.5.1 General ultimate vertical-horizontal foundation capacity envelope

The general gross ultimate vertical-horizontal foundation capacity envelope for jack-up spudcans is a two-dimensional slice of the full vertical-horizontal-moment envelope as given in A.9.3.3.2. If the spudcan moment capacity is zero (i.e. $F_M = 0$), the ultimate vertical-horizontal foundation capacity envelope is as given in Formula (A.9.3-62):

$$\left(\frac{F_H}{Q_H}\right)^2 - 16(1-a)\left(\frac{F_V}{Q_V}\right)^2\left(1-\frac{F_V}{Q_V}\right)^2 - 4a\left(\frac{F_V}{Q_V}\right)\left(1-\frac{F_V}{Q_V}\right) = 0 \quad \text{(A.9.3-62)}$$

For small embedments (in the limit as $a \rightarrow 0$), this formula reduces to Formula (A.9.3-63):

$$\left(\frac{F_H}{Q_H}\right) - 4\left(\frac{F_V}{Q_V}\right)\left(1-\frac{F_V}{Q_V}\right) = 0 \quad \text{(A.9.3-63)}$$

where Q_V is taken to be equal to the gross ultimate vertical foundation capacity of the soil beneath the spudcan (achieved during preloading), evaluated as described in A.9.3.2.2 to A.9.3.2.6, and Q_H as defined in A.9.3.3.2.

A.9.3.5.2 Ultimate vertical-horizontal foundation capacity envelopes for spudcans in sand

The yield surface used for checking the vertical-horizontal foundation capacity of spudcans in sand is presented in A.9.3.5.1.

The sliding failure envelope used for checking the sliding capacity of a spudcan in sand is as given in Formula (A.9.3-64):

$$Q_{Hs} = F_V \tan(\delta) + 0{,}5\gamma'(k_p - k_a)(h_1 + h_2) A_s \quad \text{(A.9.3-64)}$$

where

F_V is the gross vertical force acting on the soil beneath the spudcan due to the assessment load case F_d (see 8.8):

$F_V = V_{st} - B_S$ (with no backfill)

$F_V = V_{st} + W_{BF,o} + W_{BF,A} - B_S$ (with backfill) (A.9.3-65)

h_1 is the embedment depth to the uppermost part of the spudcan, (if not fully embedded $h_1 = 0$);

h_2 is the spudcan tip embedment depth;

k_a is the active earth pressure coefficient (for $s_u = 0$), $k_a = \tan^2(45 - \phi'/2)$;

k_p is the passive earth pressure coefficient, $k_p = 1/k_a$;

δ is the steel/soil friction angle in degrees:

$\delta = \phi' - 5°$ (for a flat-bottom spudcan, $\beta = 180°$),

$\delta = \phi' - 0{,}5\,(\beta - 170°)$ (for $170° < \beta < 180°$), (A.9.3-66)

$\delta = \phi'$ (for a conically shaped spudcan, $\beta \leq 170°$)

where

β is the effective cone angle in degrees (see Figure A.9.3-3);

ϕ' is the effective angle of internal friction for sand in degrees.

A.9.3.5.3 Ultimate vertical-horizontal foundation capacity envelopes for spudcans in clay

The yield surface used for checking the vertical-horizontal foundation capacity for spudcans in clay for $F_V > 0,5\ Q_V$ is presented in A.9.3.5.1 and for $F_V < 0,5\ Q_V$ in A.9.3.3.3.

The sliding capacity, Q_{Hs}, in clay can be assumed to be Q_H as defined in A.9.3.3.2.

A.9.3.5.4 Ultimate vertical-horizontal foundation capacity envelopes for spudcans on layered soils

The foundation capacity of layered soils can be determined using the principles of limiting equilibrium analysis or the finite element method. Alternatively, the formulae given in A.9.3.5.2 and A.9.3.5.3 can be used to make a conservative estimate of the ultimate vertical-horizontal capacity relationship for layered soils by considering failure through the weakest zones in such a soil profile.

A.9.3.6 Acceptance checks

A.9.3.6.1 General

Figure A.9.3-20 shows the overall approach to the foundation acceptance checks.

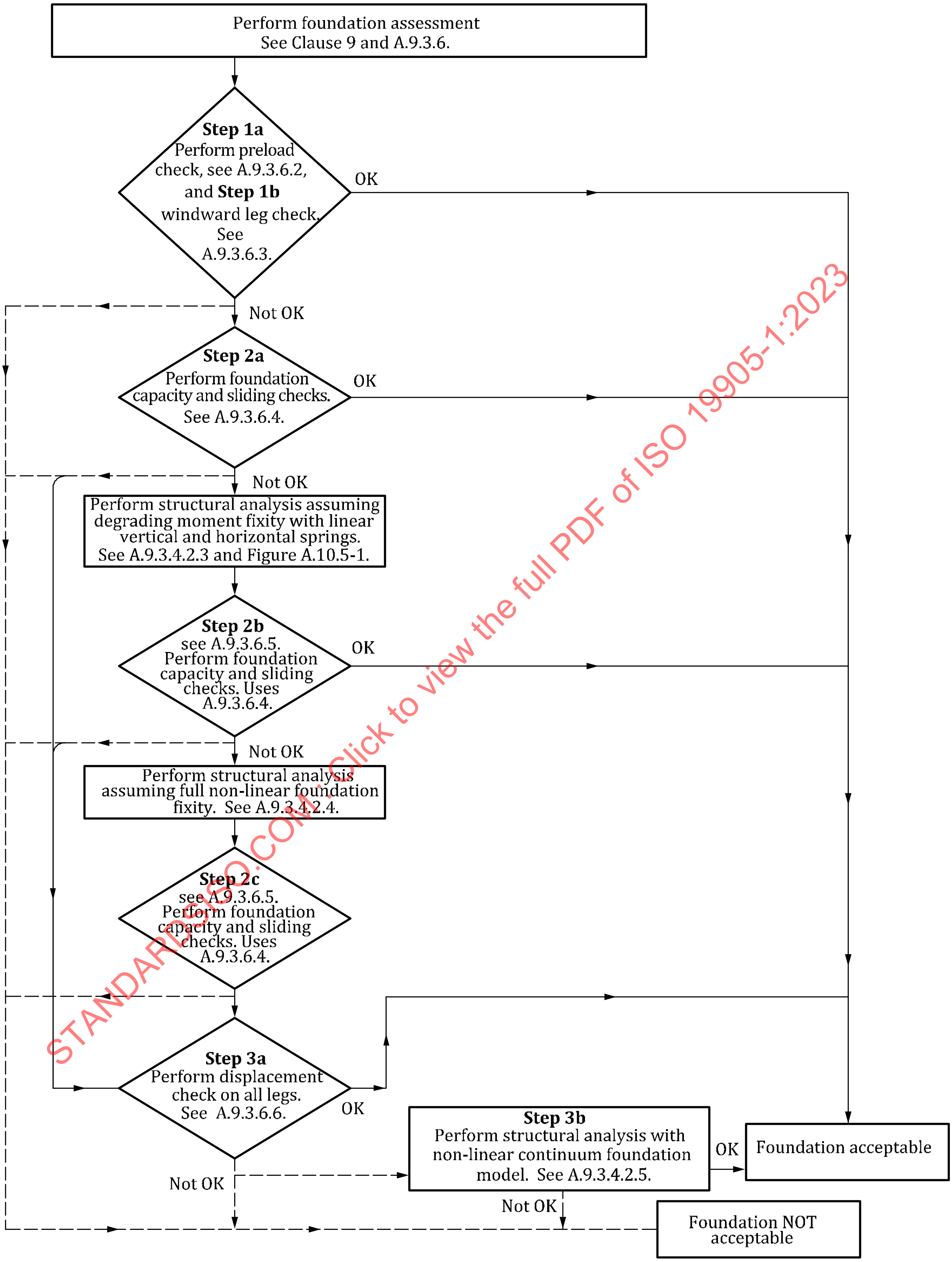


Figure A.9.3-20 — Approach to foundation acceptance checks

A.9.3.6.2 Level 1, Step 1a — Ultimate bearing capacity check for vertical loading of the leeward leg–- preload check (pinned spudcan)

The preload check should be applied only when the horizontal force on the leeward leg spudcan, F_H, is no greater than F_{H1} (see Table A.9.3-7) and when the forces are determined from an analysis model with pinned condition for all spudcans. In this case, the maximum gross vertical force F_V should conform with the limit given in the applicable Formula (A.9.3-67) or Formula (A.9.3-68):

$$F_V \leq v_{Lo} / \gamma_{R,PRE} - B_S \quad \text{(with no backfill)} \tag{A.9.3-67}$$

$$F_V \leq v_{Lo} / \gamma_{R,PRE} + W_{BF,o} - B_S \quad \text{(with backfill)} \tag{A.9.3-68}$$

where

$\gamma_{R,PRE}$ is the preload resistance factor, $\gamma_{R,PRE} = 1,10$;

$W_{BF,o}$ is the submerged weight of the overburden on top of the spudcan from backfill (backflow and infill) that is predicted to occur during preloading;

F_V is the gross vertical force acting on the soil beneath the spudcan due to the assessment load case F_d (see 8.8) as given in Formula (A.9.3-69):

$$F_V = V_{st} - B_S \quad \text{(with no backfill)}$$

$$F_V = V_{st} + W_{BF,o} + W_{BF,A} - B_S \quad \text{(with backfill)} \tag{A.9.3-69}$$

V_{st} is the vertical force applied to the spudcan due to the assessment load case F_d (see 8.8). This includes quasi-static contributions due to factored actions, and contributions from dynamic response, as appropriate, in accordance with the procedures of Clause 10, and also includes leg weight and water buoyancy but excludes the submerged weight of backfill ($W_{BF,o} + W_{BF,A}$) and the soil buoyancy of the spudcan below the bearing area B_S;

$W_{BF,A}$ is the submerged weight of any backflow and infill that is predicted to occur after the maximum preload has been applied and held.

Table A.9.3-7 — Limiting horizontal capacity, F_{H1}, for Step 1a bearing capacity check

Soil type	Embedment	Limiting horizontal capacity, F_{H1} for Step 1a to apply
Sand	Partial	$[0,1 – 0,07 (B/B_{max})^2] Q_{Vnet}$
	Full	$0,03 Q_{Vnet}$
Clay	Any	$0,03 Q_{Vnet}$

NOTE 1 The constants in Formulae (A.9.3-62) and (A.9.3-63) include the effects of $\gamma_{R,PRE} = 1,10$. The limiting horizontal capacity, F_{H1}, for Step 1a was determined from the intersection between unfactored vertical-horizontal bearing capacity envelope and maximum allowable gross vertical reaction, $Q_{V,max}$, with some reduction applied for conservatism. Assuming $a = 0$, the limiting horizontal capacity, F_{H1}, corresponding to maximum vertical capacity, $Q_{V,max}$, can be computed from Formula (A.9.3-70):

$$F_{\mathrm{H1}} = 4\left[1 - \frac{(\gamma_{\mathrm{R,PRE}} - 1)\, V_{\mathrm{Lo}}}{\gamma_{\mathrm{R,PRE}}\, Q_{\mathrm{V}}}\right]\left[\frac{(\gamma_{\mathrm{R,PRE}} - 1)\, V_{\mathrm{Lo}}}{\gamma_{\mathrm{R,PRE}}\, Q_{\mathrm{V}}}\right] Q_{\mathrm{H}} \qquad \text{(A.9.3-70)}$$

For $\gamma_{\mathrm{R,PRE}} = 1{,}10$, Formula (A.9.3-70) can be approximated by $F_{\mathrm{H1}} \approx 0{,}33\ v_{\mathrm{Lo}}\ Q_{\mathrm{H}}/Q_{\mathrm{V}}$ and is equivalent to $F_{\mathrm{H1}} \approx 0{,}04\ Q_{\mathrm{Vnet}}$ for shallow penetrations in sand. Conservatively, 0,03 was used for the limits given in Table A.9.3-7 (see also NOTE 2).

NOTE 2 For shallow spudcan penetrations and vertical reaction of 0,9 Q_{Vnet}, the available unfactored horizontal capacity is approximately 0,04 Q_{Vnet}. If the horizontal reaction exceeds 0,04 Q_{Vnet}, additional penetration can occur. The use of 0,03 Q_{Vnet} in the check, therefore, includes a level of conservatism. If the spudcan is fully embedded, the additional penetration can be significant. Additional penetration can increase the soil resistance but, to increase the horizontal capacity to 0,1 v_{Lo}, the additional penetration is about 10 % of the spudcan diameter and outside tolerable limits. Conversely, where the spudcan is partially embedded (i.e. when the maximum spudcan bearing area is not mobilized), any additional penetration results in a significant increase of bearing capacity due to the rapid increase in the bearing area. An increase in embedded area of approximately 10 % increases the vertical bearing capacity such that, simultaneously, the horizontal foundation capacity increases to 0,1 v_{Lo}.

NOTE 3 For partial spudcan penetration in sand, Q_{Vnet} can be taken as being equal to v_{Lo} for the purposes of the Step 1a check.

A.9.3.6.3 Level 1, Step 1b — Check of the windward leg — Pinned spudcan

The windward leg check should be applied only when the horizontal force on the windward leg spudcan, F_{H}, is no greater than F_{H1} (see Table A.9.3-7). In this case, the sliding stability of the windward leg is checked by ensuring that the vertical reaction complies with Formula (A.9.3-71):

$$F_{\mathrm{V}} > (1 - 1/\gamma_{\mathrm{R,PRE}})\ Q_{\mathrm{V}} \qquad \text{(A.9.3-71)}$$

where $\gamma_{\mathrm{R,PRE}}$ is the preload resistance factor, $\gamma_{\mathrm{R,PRE}} = 1{,}10$.

In the case of a sand foundation, this check is valid for sand friction angle $\phi' \geq 25°$. For friction angles $\phi' < 25°$, the sliding check in Step 2 should be performed.

A.9.3.6.4 Level 2, Step 2a — Foundation capacity and sliding check — Pinned spudcan

A.9.3.6.4.1 Step 2a — Foundation capacity check

The combined vertical and horizontal forces on all spudcans should be checked against the corresponding factored foundation bearing capacity envelope and the factored sliding capacity envelope, see Figure A.9.3-21. Each leg's spudcan reaction forces should lie within the corresponding factored vertical-horizontal foundation capacity envelope. In addition, the reaction forces should lie within the corresponding factored sliding capacity envelope. The following describes the construction of the factored vertical-horizontal foundation capacity envelope and the foundation capacity check for Step 2a which is also applicable to Step 2b. The construction of the sliding failure surface and the foundation sliding check is described in A.9.3.6.4.2.

A reduction in the ultimate vertical bearing capacity, Q_{V}, of a spudcan foundation occurs when it is simultaneously subjected to a horizontal force, F_{H}, and a moment, F_{M}. The latter is ignored in Step 2a analyses as the spudcans are considered to be pinned.

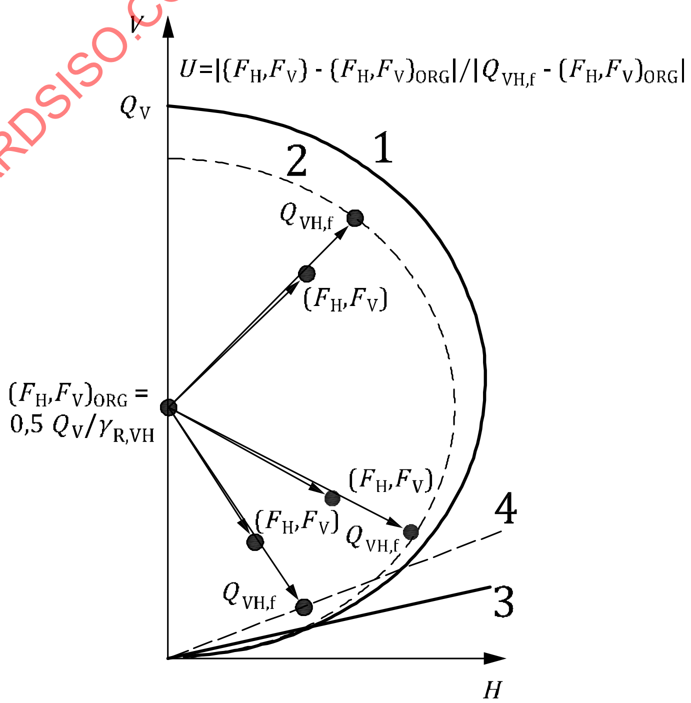
The vertical-horizontal foundation capacity for sands and clays can be generated according to A.9.3.5 and the spudcan reactions should be evaluated for each spudcan. To obtain the factored vertical-horizontal bearing envelope, the vertical-horizontal capacity envelope is scaled by the resistance factor, $\gamma_{\mathrm{R,VH}}$, from the point of zero net reaction, i.e. ($F_{\mathrm{H}} = 0$, $F_{\mathrm{V}} = W_{\mathrm{BF,o}} - B_{\mathrm{S}}$). In effect, the envelope is shrunk towards this scaling origin.

A measure of the foundation utilization (see Clause 13) can be obtained by assessing the proximity of the loading point (F_H, F_V) to the factored vertical-horizontal bearing capacity envelope, see Figure A.9.3-21. When making the check, the magnitude of the vector to the loading point should be compared against the magnitude of the vector to the factored vertical-horizontal bearing capacity envelope. The origin of the vectors is arbitrary; however, for consistency and to help produce a meaningful value of the resulting utilization, the origin of the vectors $(F_H, F_V)_{ORG}$ should be taken on the vertical capacity axis (at zero shear) at 0,5 $Q_V/\gamma_{R,VH}$ (see Figure A.9.3-21). Accordingly, each spudcan foundation should satisfy the capacity check given in Formula (A.9.3-72):

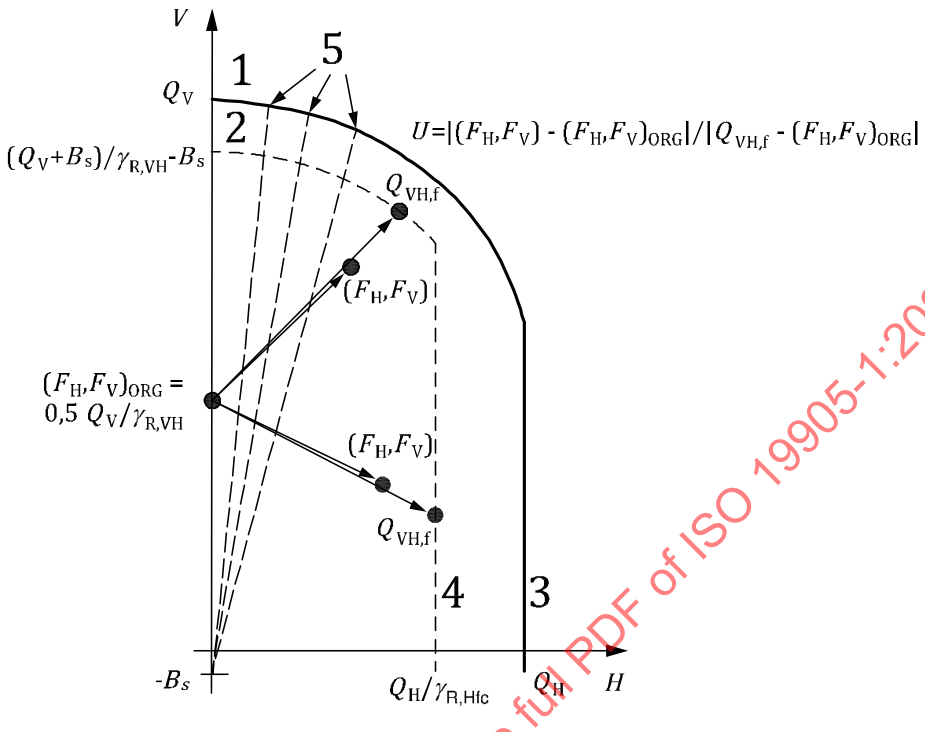
$$| (F_H, F_V) - (F_H, F_V)_{ORG} | \leq | Q_{VH,f} - (F_H, F_V)_{ORG} | \quad \text{(A.9.3-72)}$$

where

(F_H, F_V)	is the environmental response point (determined from factored actions);
$(F_H, F_V)_{ORG}$	is the origin used for establishing the utilization; this should be taken as $H = 0{,}0$; $V = 0{,}5Q_V/\gamma_{R,VH}$;
Q_V	is the gross ultimate vertical foundation capacity;
$Q_{VH,f}$	is the point where the vector originating from $(F_H, F_V)_{ORG}$ and passing through (F_H, F_V) intersects the applicable factored vertical-horizontal capacity surface as defined in A.9.3.6.4.1 or the factored sliding capacity as defined in A.9.3.6.4.2. The factored vertical-horizontal capacity surface is derived by dividing the coordinates of the applicable surface from A.9.3.5 by the resistance factor $\gamma_{R,VH}$ with respect to the point of zero net reaction $(0, W_{BF,o} - B_S)$;
$\gamma_{R,VH}$	is the partial resistance factor for vertical-horizontal foundation bearing capacity, $\gamma_{R,VH} = 1{,}10$;
|...|	represents the vector magnitude.

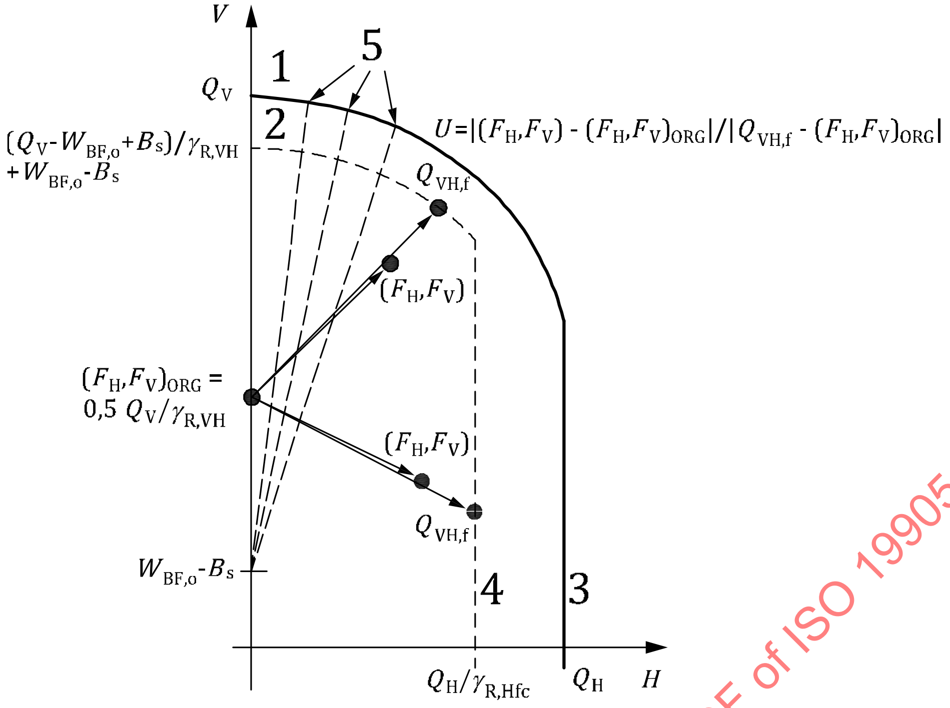


a) Sand



b) Clay with spudcan buoyancy and no backfill

Figure A.9.3-21 — Vertical-horizontal foundation capacity envelopes (1 of 2)



c) Clay with spudcan buoyancy and backfill

Key

1	vertical-horizontal foundation capacity
2	factored vertical-horizontal foundation capacity (coordinates multiplied by $1/\gamma_{R,VH}$) relative to the scaling origin as defined
3	sliding capacity (see A.9.3.6.4.2)
4	factored sliding capacity (unfactored horizontal sliding capacity coordinate multiplied by $1/\gamma_{R,Hfc}$)
5	vectors indicating origin for construction of the factored V-H bearing capacity envelope
|...|	represents the vector magnitude
H	horizontal reaction or horizontal capacity
Q_V	gross ultimate vertical foundation capacity (with zero horizontal load)
$Q_{VH,f}$	point where the vector originating from $(F_H, F_V)_{ORG}$ and passing through (F_H, F_V) intersects the factored vertical-horizontal capacity surface as defined in A.9.3.6.4.1 or the factored sliding capacity surface as defined in A.9.3.6.4.2
U	utilization for environmental response point (F_H, F_V) as given in A.9.3.6.4
V	vertical reaction or vertical capacity
$\gamma_{R,VH}$	partial resistance factor for foundation (bearing) capacity
$\gamma_{R,Hfc}$	partial resistance factor for horizontal (sliding) capacity

Figure A.9.3-21 — Vertical-horizontal foundation capacity envelopes (2 of 2)

A.9.3.6.4.2 Step 2a — Foundation sliding check

In Step 2a, the spudcan foundations should also be assessed using the following sliding check, since the factored sliding failure surface can lie within the factored vertical-horizontal bearing capacity envelope as in A.9.3.6.4.1, see figure A.9.3-21. The same procedure also applies for Step 2b.

The horizontal capacity of the foundations of all legs should be checked for the horizontal forces on the spudcans, F_H, in association with the gross vertical force F_V.

NOTE The most onerous case is likely to be with a single windward leg, the minimum variable load and the centre of gravity offset to leeward.

The foundation should satisfy the capacity check given in Formula (A.9.3-73):

$$| (F_{H}, F_{V}) - (F_{H}, F_{V})_{ORG} | \leq | Q_{VH,f} - (F_{H}, F_{V})_{ORG} | \quad \text{(A.9.3-73)}$$

where

$Q_{VH,f}$ is the point where the vector originating from $(F_{H}, F_{V})_{ORG}$ and passing through (F_{H}, F_{V}) intersects the applicable factored sliding capacity surface derived by dividing the horizontal coordinates of the applicable surface Q_{Hs} from A.9.3.5.2 (sand) or A.9.3.5.3 (clay) by the resistance factor, $\gamma_{R,Hfc}$;

Q_{Hs} is the foundation sliding capacity; see A.9.3.5.2 (sand) or A.9.3.5.3 (clay);

$\gamma_{R,Hfc}$ is the partial resistance factor for horizontal foundation capacity where $\gamma_{R,Hfc}$ = 1,25 for both sand (based on drained conditions and effective stress) and clay (based on undrained conditions and total stress);

|...| represents the vector magnitude.

A.9.3.6.5 Level 2, Steps 2b and 2c — Foundation capacity and sliding check — Spudcan with moment fixity and vertical and horizontal stiffness

Step 2b and 2c foundation analyses inherently ensure conformity with the unfactored foundation yield surface, except that in a Step 2b analysis conformity is no longer assured when the moment fixity has reduced to zero, i.e. the spudcan has become pinned.

The capacity checks to undertake in a Step 2b assessment are identical to those undertaken for Step 2a in which vertical-horizontal capacity and sliding capacity for sands and clays can be generated in accordance with A.9.3.5 and the spudcan reactions are evaluated for each spudcan. If the vertical and horizontal reactions from the response analysis (which has accounted for spudcan moment fixity with stiffness reduction) lie within the factored foundation capacity envelopes, the foundation is satisfactory.

A Step 2c analysis implicitly includes a check on conformity with the unfactored foundation yield surface. When the frictional sliding surface intersects the foundation capacity envelope, sliding can occur before the response reaches the yield surface. When this sliding effect is included in the response analysis, no further Level 2 checks are required. When this sliding effect is not included, a sliding check should be undertaken in accordance with A.9.3.6.4.2. In all cases, the Level 3, Step 3a displacement check should be performed.

A.9.3.6.6 Level 3, Steps 3a and 3b — Displacement check — Settlements resulting from exceedance of the foundation capacity

Vertical settlement and/or sliding of a spudcan can occur if the forces on the spudcan due to the extreme event are outside the yield interaction surface computed for the spudcan at the penetration achieved during installation. Such settlements often result in a gain in capacity through expansion of the yield interaction surface. However, the integrity of the foundation can decrease in the situation where a potential punch-through exists, e.g. where dense sand overlies soft clay. More thorough analyses should be performed for such cases and for the complex and/or potentially dangerous foundation conditions listed in A.9.3.2.5 and A.9.3.2.6.

A Step 3a check can be accomplished by identifying the “equivalent” preload level that would be required to expand the *V-H* yield surface used in Step 2 such that the factored capacity exceeds the forces on the spudcan. The added penetration associated with this “equivalent” preload is calculated using each of the three predicted load-penetration curves [using the best estimate, a high representative value and a low representative value of soil strength profile and separate global response analyses as appropriate; see A.9.3.2.1.1 b)]. If any of these three additional penetrations is significant, the effects on the spudcan foundation and the structure should be evaluated and the procedure iterated to establish whether the consequences of the displacement on the other utilization checks are acceptable.

A Step 3a check can also be performed when the Level 2a or 2b sliding or capacity check of a windward leg is not satisfied, or is no longer satisfied due to the additional penetration of a leeward leg as described above. In the case of a windward leg, sliding can occur when the factored load exceeds the factored capacity resulting in redistribution of the horizontal reaction to the leeward leg foundations. This effect can be assessed by limiting the factored horizontal reaction to the factored sliding limit (dependent on F_V) and iteratively determining the redistribution of loads and the associated non-linear displacement of the structure. The effects on all spudcan foundations and the structure should be evaluated and the procedure iterated to establish whether the consequences of the displacement on all the other utilization checks are acceptable, including the foundation capacity of the other legs.

A Step 3b analysis inherently includes a check on the direct consequences of spudcan displacement. Therefore, no foundation checks are required, although it should be shown that the results are not sensitive to the load-penetration assumptions, i.e. that small changes in the forces on the spudcan or assumed soil strength do not lead to large increases in penetration.

When assessing the acceptability of displacements, due consideration should be given to operational limitations, e.g. jacking operations to level the unit and re-establish a safe hull elevation or to depart the site. The limits are dependent upon the jack-up and the configuration at the site.

A.9.3.6.7 Foundation settlement not specifically addressed elsewhere

Settlement of the spudcans should be estimated and checked. If necessary, corrective actions should be taken. The settlements of installed spudcans can be assessed from a combination of:

- elastic settlements;
- consolidation settlement;
- settlements due to cyclic loading;
- settlements due to seabed instability.

The elastic settlements and consolidation settlements can be calculated using conventional analytical or numerical geotechnical models (see ISO 19901-4). The elastic settlements occur concurrently with applied actions and can be calculated as function of the basic elastic soil properties (ν and G) and the applied actions. The consolidation settlements of cohesive soils can be calculated using conventional models accounting for time effects.

Cyclic environmental actions or operational vibrations can induce further settlements. Special attention should be given to cyclic loading in silty sand or silt. Cyclic loading can also involve a soil strength reduction. This can induce settlements due to bearing failure.

Seabed instability due to scour or gas seeps involves a decrease in the effective bearing capacity. This can induce settlements due to local bearing failure.

The settlements should be checked regularly. If necessary, level adjustments should be made or protective measures against scour development should be taken (see A.9.4.7).

A.9.4 Other considerations

A.9.4.1 Skirted spudcans

Skirts are added to spudcans to provide additional foundation capacity and stiffness compared to conventional conical spudcan geometries.

Within the skirt, the typical geometry of the underside of a skirted spudcan is either relatively flat or conical. In some cases, the leg chords may protrude below the skirt tip and achieve first contact with the sea floor, thus protecting the skirt whilst going on and off site. In the case of skirted spudcans with a flat underside, a level and undisturbed sea floor surface is required in order to minimize the potential for eccentricity of the foundation reaction.

In order to realise the maximum benefit from using a skirted spudcan, the underside of the skirted spudcan should achieve full contact with the sea floor surface. Calculations should be performed to determine the penetration resistance of the skirt, including any bulkheads and internal or external stiffeners and, hence, whether the applied preload is sufficient to ensure that full contact is achieved.

Methods for calculating the tip and skin friction components of the skirt penetration resistance are described in DNV-RP-C212 (DNV 2021f). In cases where the skirt tip has a greater thickness than the rest of the skirt, consideration should be given to the potential for a gap to form above the skirt tip during penetration into the seabed, especially in cohesive soils.

If the penetration resistance exceeds the available preload spudcan reaction and partial penetration of the skirt occurs, consideration can be given to measures such as applying suction for increasing the penetration or infilling the resulting void within the skirt with suitable material introduced through valved pipes that penetrate into the skirt void. If, after preloading, the skirt is partially penetrated, the assessment should be revised to determine the consequences, including a consideration of the strength of the skirt.

Consideration should also be given to the effects of compaction and/or consolidation of the soils within the skirt or any infill material used during preloading.

If the voids within the skirt are not completely infilled, consideration should be given to the effect of movement of the enclosed seawater within the skirt due to spudcan rotation, especially for compartmentalized skirts in cohesionless soils where “piping” can occur due to flow of the enclosed water around stiffener plates or bulkheads.

Once full contact has been achieved, the vertical bearing capacity of the skirted spudcan essentially corresponds to that of an embedded footing. As the soil within the spudcan skirt is effectively part of the spudcan, the weight of the enclosed soil plug should be incorporated in the penetration resistance calculations.

At sites with relatively strong soils and where the underside of the skirted spudcan is flat, or any remaining voids are filled with an appropriate material, the ultimate bearing capacity of the foundation can be significantly greater than the applied preload. Guidance is provided in E.4 for the use of foundation capacities that are greater than those developed by preloading.

The bearing capacity envelopes appropriate for skirted footings have been the subject of much research; see Dean et al. (1995)[56], Bransby and Randolph (1998[27], 1999[32]), Bransby and Yun (2009)[33], Cassidy et al. (2004a)[45], Eide et al. (1996)[69], Gourvenec (2003)[82], (2008)[83], Gourvenec and Randolph (2002)[84], Kellezi et al. (2005a[116], 2007[118], 2008[119]), Leland et al. (1994)[126] and Svanø and Tjelta (1996)[176]. The skirted spudcan has generally been modelled as a solid footing, however, care is warranted before making such an assumption as weaker soil from the seabed surface trapped within the spudcan skirt can influence the failure mechanisms developed, reducing the additional capacity available; see Bransby and Yun (2009)[33].

When full spudcan-seabed contact is achieved, the embedment of the skirted spudcan can permit the use of elastic foundation stiffness depth factors corresponding to a solid footing as described in Bell (1991)[23].

The extraction resistance for a skirted spudcan can be substantial; consequently, skirted spudcans are not usually employed at sites where soil backfill can occur on top of the spudcans. Extraction can be assisted through the use of drainage and/or the application of water pressure within the skirt in order to minimize the development of suction within the soil below the spudcan.

Soil can remain within the skirts after extraction of a skirted spudcan from a site with cohesive soils, which can influence the penetration response during subsequent installations.

A.9.4.2 Hard sloping strata

A hard, sloping stratum can be created by a sand wave, sand bank, scour around a platform, buried geomorphic features such as channels, footprints produced by previous jack-up emplacements, human-related seabed activity, or a combination of the above. Such slopes can cause eccentricity in the spudcan reaction, which can lead to emplacement and removal difficulties, particularly for leg designs with slender braces, as in the following examples.

— The eccentric reaction can result in a significant leg bending moment in the region of the hull. Where this bending moment is reacted by the leg guides, the resulting large shear force can overstress the leg members.

— If a fixation system (rack chocks) is employed at the leg-to-hull interface, the bending moment present at the time when the fixation system is engaged is locked into the leg. If the eccentricity of the spudcan reaction is subsequently exacerbated (e.g. by scouring around the spudcan), then the effective leg bending moment in the region of the hull can increase. When the fixation system is later disengaged, the redistribution of the moment in the leg for the revised support condition provided by the pinions and guides can cause overstress.

Anticipated installation-induced stresses should be considered in the site-specific assessment (see 5.4.8). The foundation reactions should be assessed against bearing and sliding capacities of the sloping hard stratum.

Consideration can be given to the potential benefit of seabed preparation prior to emplacement of the jack-up.

A.9.4.3 Footprint considerations

Surface or buried footprints from prior jack-up operations in the proposed field can cause eccentric reactions or lateral movement of the spudcan. One preventive approach is avoidance (i.e. positioning spudcans at some minimum distance away from the footprints) while mitigations include working the legs, leg stomping, seabed remediation, etc.

Information on spudcan-footprint interaction can be found in Stewart and Finnie (2001)[173], Cassidy et al. (2009)[48], Dean and Serra (2004)[55], Jardine et al. (2001)[111], Teh et al. (2006)[181], Gaudin et al. (2007)[78], Gan et al. (2008)[77] and Foo et al. (2003)[73].

A.9.4.4 Leaning instability

A lower bound estimate of the leaning stability can be obtained using the theory of Hambly (1985)[87]. However, such estimates have proven to be generally conservative due to the omission of beneficial effects such as spudcan fixity and lateral soil resistance on the legs.

At locations where a relatively large proportion of the leg length is below the hull, a potentially unsafe condition (comparable to a punch-through situation) can occur. The potential for such incidents can be mitigated if appropriate installation procedures are adopted. These can, for example, include preloading the spudcans individually.

A.9.4.5 Leg extraction difficulties

Leg extraction difficulties can be caused by conditions including the following:

- deeply penetrated spudcan in soft clay or loose silt;
- skirted or caisson-type spudcan where uplift resistance can be greater than the installation reaction;
- sites where the soil exhibits increased strength with time.

A jack-up pulls its legs from the seabed by lowering the hull into the water, thereby generating a buoyant uplift force and inducing tensile forces in the legs. The force required to extract the leg is affected by several factors, including the nature of the soils, the depth of penetration, the geometry of the spudcan and whether soil backfill has occurred. The force available for leg extraction is frequently less than the force applied during installation. Where significant leg penetrations are attained, it is not uncommon for pulling of the legs to take several days, or in some cases much longer.

Where leg extraction problems are predicted, a warning should be included in the site-specific assessment report.

Potential mitigations include jetting and/or excavation of the surface soils. However, these measures can alter soil strength and the seabed topography, which can affect the future emplacement of jack-ups at the same site.

Further details can be found in Byrne and Cassidy (2002)[38], Craig (1998)[49], Craig and Chua (1990b)[51], Craig et al. (2002)[52], Erbrich (2005)[70] and Purwana et al. (2005)[149].

A.9.4.6 Cyclic mobility, liquefaction and liquefaction-induced lateral flow

A.9.4.6.1 General

Liquefaction and/or cyclic mobility can occur as a result of low frequency excitation (waves) and higher frequency excitation (such as earthquakes and ice-flow-induced vibrations).

Wave-induced liquefaction of the seabed soils can occur. Dean (1991)[54] presents approximate methods for estimating settlements of submerged foundations subjected to low frequency, non-earthquake, time dependent loading (these are not discussed further here).

General guidance on the assessment of the potential for liquefaction and/or cyclic mobility from earthquakes is given by Kramer (1996)[121]; Idriss and Boulanger (2004)[104]; Kayen et al. (2013)[115]; Robertson (2015)[156]; and Boulanger and Idriss (2016)[30]. Large strains and degraded shear strength can also occur in very soft clay due to ground shaking (Boulanger and Idriss (2007)[28]) (this is not discussed further here).

Assessment of the potential for, and the effects of, earthquake induced liquefaction on the jack-up involves addressing

a) Liquefaction-induced lateral flow in the vicinity of the jack-up, Edgers and Karlsrud (1982)[68] and Stewart and Kwok (2008)[171],

b) Liquefaction initiation in the zone of influence of the spudcan foundation,

c) Spudcan displacements and/or loss of support induced by liquefaction, and

d) Actions on the leg from liquefied zones.

The risk of earthquake induced liquefaction at the specific site can be assessed by simplified (A.9.4.6.5) or detailed (A.9.4.6.6) procedures.

The liquefaction assessment discussed in A.9.4.6.3 – A.9.4.6.6 is applicable only to siliceous sands and non-plastic silts and should not be applied to other soil types, such as carbonate soils, see 9.4.10 and A.9.4.10.

It is emphasized that the simplified methods in A.9.4.6.5 are based on the 1 000 year earthquake and free-field conditions, i.e. not considering the effects of the presence of the jack-up. This is a significant limitation of these methods and potentially unconservative. Unless these methods indicate that the likelihood of liquefaction is low, a detailed assessment should be performed based on the ALE event and include the presence and dynamic response of the jack-up.

A.9.4.6.2 Glossary

A.9.4.6.2.1 Cyclic mobility

Temporary or permanent deformation of soil that occurs when cyclic loading creates (excess) pore pressure.

A.9.4.6.2.2 Liquefaction

Liquefaction is a phenomenon in which the strength and/or stiffness of a soil is reduced due to an increase in pore-water pressure caused by earthquake shaking or other cyclic loading.

A.9.4.6.2.3 Liquefaction potential

Susceptibility of the soil to the onset of liquefaction under a reference earthquake motion.

A.9.4.6.2.4 Free-field

Seabed not subject to the effect of geotechnical works or structures.

A.9.4.6.2.5 Liquefiable

Susceptible to liquefaction or cyclic mobility for a contemplated means of initiation.

A.9.4.6.2.6 Liquefaction-induced lateral flow

Liquefaction leading to mass flow.

A.9.4.6.3 Assessment of earthquake-induced liquefaction

Where spudcans are founded on liquefiable soils, actions from an earthquake can cause large movements of a spudcan in both vertical as well as horizontal directions. The potential consequences on jack-ups can include:

a) loss or reduction of bearing capacity;

b) excessive settlement;

c) differential settlement among the legs;

d) lateral spreading of the seabed resulting in either uniform or differential leg displacements;

e) loss or reduction of spudcan rotational stiffness.

Consideration should be given to possible effects of additional actions that can occur from liquefied soil surrounding or above the spudcans.

To assess the potential consequences of liquefaction to the jack-up on a site, the following three clauses provide possible methods with increasing level of complexity:

1) Identify whether soils on the site are susceptible to liquefaction (A.9.4.6.4).

2) Perform a free-field liquefaction triggering assessment based on simplified procedures or non-linear site response analysis (A.9.4.6.5).

3) Assess liquefaction and its consequences in a detailed analysis including the jack-up (A.9.4.6.6).

An assessment of the potential for triggering of liquefaction can be performed by evaluating the ratio of Cyclic Stress Ratio and Cyclic Resistance Ratio. This can be done by

— Using simple formulae with available geotechnical data, such as CPT and shear wave velocity data (simplified calculations A.9.4.6.5.2), or preferably by

— Using a non-linear Site Response Analysis (see for example Stewart and Kwok (2008)[171]) including pore pressure development (more detailed time history analyses A.9.4.6.5.3).

A detailed assessment should be performed as part of an ALE assessment (A.10.7.4) unless the likelihood of triggering of liquefaction is assessed as low.

More information can be found in ISO 19901-2, ISO 23469 and EN 1998-5:2004.

A.9.4.6.4 Assessment of site susceptibility to liquefaction

The first consideration in a hazard evaluation is the assessment of site's susceptibility to liquefaction as can be determined by historical, geological, compositional and stress state criteria, see Kramer (1996)[121].

An evaluation of the site susceptibility to liquefaction should be made when the foundation soils include layers or thick lenses of loose to medium dense sands and non-plastic silts. Case histories indicate that liquefaction usually occurs within a depth of 20 m or less (Kramer (1996)[121], Boulanger and Idriss (2016)[30]).

For many years liquefaction was thought to be limited to sands. However, liquefaction of non-plastic silts has been observed and therefore plasticity characteristics are of significance. Coarse silts which are non-plastic and with bulky particles are at risk of liquefaction; clays are non-susceptible. Well graded soils are generally less susceptible than poorly graded soils and soils with rounded grains have a greater tendency to contract and therefore liquefy under cyclic loading. Dense sands are less susceptible to cyclic loading and to the accumulation of strains and displacements.

According to Boulanger and Idriss (2006)[27], soils with fines content less than 35% can be susceptible to liquefaction. The liquefaction hazard may be neglected (Eurocode 8 [EN 1998-5:2004]) if peak ground acceleration (at the sea floor) is less than 1,5 m/s^2 (0,15g) and at least one of the following conditions is fulfilled:

a) the soil has a clay content greater than 20% with plasticity index greater than 10;

b) the strata has cone tip resistance, normalized with respect to atmospheric pressure, greater than 180 and shear wave velocity greater than 250 m/s.

Historical assessment is useful for seismic studies, since it is known that susceptibility is dependent on magnitude and proximity to the earthquake epicentre. Geological assessment is useful since soils that are loose and of uniform grain size are most susceptible. Therefore, fluvial soils of Holocene age are more susceptible than Pleistocene deposits. In general, older deposits are less susceptible than younger deposits. Furthermore, the applied cyclic shear stresses from historical wave loading and low-level earthquakes over time can reduce the risk of liquefaction, Sassa and Sekiguchi (1999)[161], Clukey et al. (1989)[53].

A.9.4.6.5 Simplified free-field assessment of liquefaction

A.9.4.6.5.1 General

Free-field liquefaction can occur when the earthquake induced Cyclic Stress Ratio exceeds the Cyclic Resistance Ratio of a soil layer. An initial screening assessment of the margin against triggering of liquefaction can be performed based on simplified methods. Two approaches are given in the following subclauses.

The simplified methods provide a ratio of normalized cyclic resistance ratio λ_{CRR} over normalized cyclic stress ratio λ_{CSR} giving a margin against triggering of liquefaction, i.e.

$$f_{trig} = \lambda_{CRR}/\lambda_{CSR}$$

A factor f_{trig} larger than 1,0 typically implies no liquefaction. However, these methods are approximate (Pyke and North, 2019[152]) and should be used only as a screening tool (Pyke, 2015)[150]. Even if liquefaction is not triggered, the soil can exhibit reduction in strength and/or stiffness due to increases in pore water pressure. The effects of these increases in pore water pressure should be accounted for during evaluations of stability and settlement.

Simplified methods for evaluating liquefaction-induced liquefaction and settlement are adversely affected by the fact that cone penetration resistance provides only an indirect indication of the response of soils to cyclic loading, especially if the soils contain clayey fines. Transition and thin layer effects, the use of peak ground accelerations at the sea floor, standard depth reduction and magnitude weighting factors, and the failure to account for excess pore pressure redistribution and dissipation will add to the uncertainty in the results. External factors, such as discontinuous sand layers, the thickness of any overlying "crust", and partial saturation, are not considered in simplified methods. These various shortcomings are discussed in more detail by Pyke and North (2019)[152].

Effects from the spudcan footing, such as stresses from the structure and its dynamic response, drainage conditions, preloading, consolidation etc., can also affect liquefaction susceptibility. These effects are not included in these simplified methods.

The above limitations and effects should be considered in the assessment of earthquake-induced liquefaction. Liquefaction-induced ground deformations should also be considered.

A.9.4.6.5.2 Cyclic shear stress ratio and cyclic resistance ratio

Simplified methods Seed and Idriss (1971)[165], Idriss (1999)[101], Youd et al. (2001)[210], Zhang et al. (2002)[214], Idriss and Boulanger (2006)[105] can be used to further assess the free-field liquefaction potential within the upper part of the soil profile by estimating the normalized cyclic stress ratio (λ_{CSR}) and comparing it to the normalized cyclic resistance ratio (λ_{CRR}) computed from cone penetration test resistance (Moss et al., 2006[139]; Boulanger and Idriss, 2016[30]) or in situ shear wave velocity (Kayen et al., 2013[115]; Robertson, 2015[156]).

Possible free-field liquefaction assessment methods to calculate the ratio of λ_{CRR} over λ_{CSR} are provided in Annex E.5.

A.9.4.6.5.3 Site response analysis

A non-linear site response analysis (including softening) in combination with pore pressure development calculations can give a more realistic value of liquefaction, as discussed in Pyke and North (2019)[151]. Ideally, this will also be a bi-directional analysis which uses both horizontal components of motion as input (Pyke, 2019)[151], Hashash et al. (2010)[89], Olson et al. (2020)[143].

A.9.4.6.6 Detailed analysis

In ALE Stage 3 time-history analyses the effects of the presence and dynamic response of the jack-up are considered with the jack-up footings supported by a soil model, such as in Galanes-Alvarez et al. (2020)[76]. See also 10.10 and A.10.7.4.

Vertical settlement of the soils under the applied spudcan load should be considered for each leg of the jack-up. The differential displacements between the legs can be critical to structural stability and should be evaluated.

Detailed analyses are also recommended to evaluate "lateral spreading" if the site is located on or near sloping ground. Lateral deformations could potentially cause stability issues for the jack-up.

Consideration of lateral deformations can be important when surface layers are decoupled from deeper layers as a result of liquefaction of intermediate layers see Zhang et al. (2004)[215]. This decoupling can occur whether the sea floor is sloped or level.

A.9.4.6.7 Liquefaction-induced lateral flows

The potential for liquefaction-induced lateral flow in the vicinity of the jack-up should be considered, see A.6.5.1.

A.9.4.7 Scour

The key conditions for scour are

— hydrodynamic conditions,

— flow disturbance due to presence of an obstruction, and

— potential for erosion of the sea floor material.

For the hydrodynamic conditions, the combination of tidal and non-tidal current velocities (e.g. storm-driven currents) are key parameters, so that the effects of scour can increase rapidly during storms, particularly when the two contributions are aligned.

The maximum depth of scour adjacent to the spudcan is related to the dimensions of the obstruction introduced, e.g. the spudcan itself, the spudcan in combination with the leg structure and/or adjacent infrastructures.

Particle size has a strong influence on the erodibility; see Figure A.9.4-2. Particle sizes larger than those of the original sea floor, such as gravels and cobbles can be useful for scour protection.

Scour is more important for spudcans with limited sea floor penetration, as removal of the soil can result in the following:

— a redistribution of leg forces or loss of jack-up hull trim;

— a reduction of the bearing capacity of the foundation and seabed fixity;

— eccentricity in the spudcan reaction;

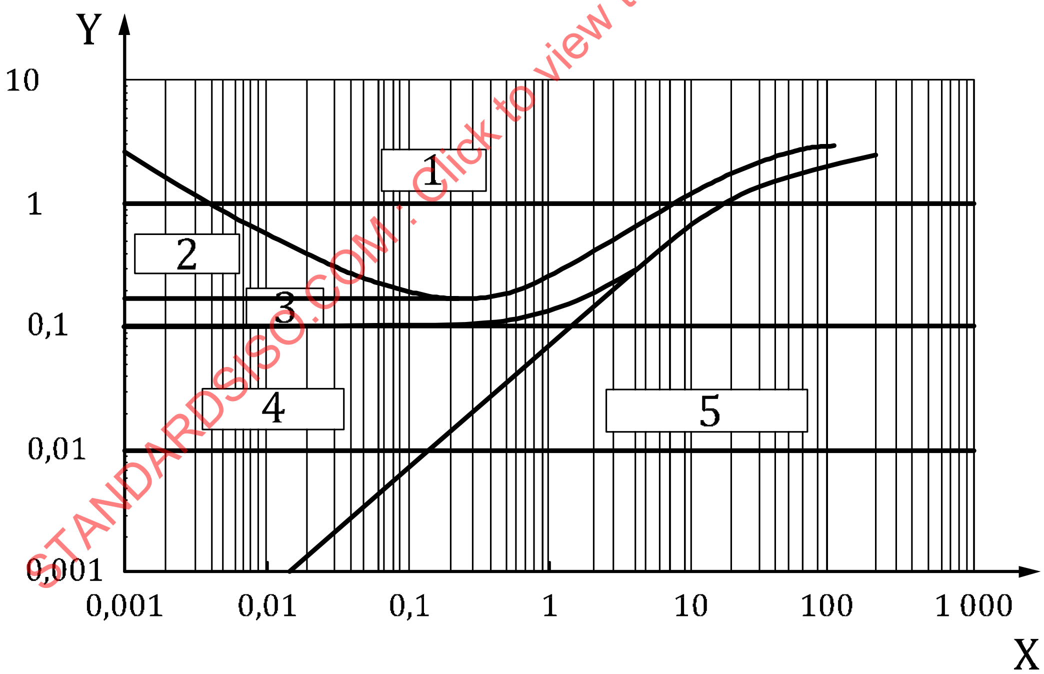
— an increase in an existing potential for punch-through in layered soils.

There is no definitive procedure for the evaluation of scour potential, but useful reference material can be found in Sweeney et al. (1988)[177]; Whitehouse (1998)[201] and Rudolph et al. (2005) [158]. Previous operational experience can help in the management of scour, either in the development of scour protection measures or of an awareness of the critical combination of tidal and non-tidal (storm driven) currents that can induce scour. Scour protection measures include the following:

a) gravel dumping prior to installation, provided the selected gravel gradation does not cause damage to the jack-up spudcans: Particularly for the larger materials, care should be taken to ensure that this activity does not adversely affect future jack-up emplacements;

b) use of frond mats, gravel bags, gravel dumping or grout mattresses after installation, the effectiveness of which can be evaluated from scour surveillance monitoring;

c) monitoring and adjusting for reduction in hull elevation.



CLAY	Fine	Medium	Coarse	Fine	Medium	Coarse	Fine	Medium	Coarse	Cobbles	Boulders
	SILT			SAND			GRAVEL				

Key

X particle size, expressed in millimetres

Y mean flow velocity, expressed in metres per second

1 erosion

2 transport/erosion

3 transport

4 sedimentation/transport

5 sedimentation

Figure A.9.4-2 — Soil particle size and seabed mobility
[after McDowell and O'Connor (1977)[137]]

A.9.4.8 Spudcan interaction with adjacent infrastructure

The interaction of the spudcans with adjacent infrastructure can be addressed with reference to the literature, e.g. Siciliano et al. (1990)[168], Stewart (2005)[171], Leung et al. (2006)[127], and Kellezi et al. (2005b)[117].

A.9.4.9 Geohazards

Certain areas of the world, including the US, require shallow geohazard surveys and publish documents that can give some useful guidance, e.g. US Department of the Interior Minerals Management Service (2008)[192] and IOGP (2009)[107]. It is important that the work is planned, performed and assured by qualified geohazard specialists to ensure that it is fit-for-purpose and meets the actual regulatory requirements of the host country. See also ISO 19901-10.

A.9.4.10 Carbonate material

No guidance is offered.

A.10 Guidance on structural response

A.10.1 Applicability

No guidance is offered.

A.10.2 General considerations

The ULS responses typically include overturning moments of the jack-up, reactions and displacements at the spudcans, horizontal deflections of the hull, the internal forces in the leg members and forces in the holding system. The responses should be obtained using appropriate combinations of functional actions, metocean or earthquake actions, and dynamic, second order and leg inclination effects with the action factors in Annex B. The application of actions is described in 8.8 and A.8.8. In 5.4.3, it is required that the analysis be carried out for a range of headings with respect to the jack-up such that the most onerous loading(s) for each item in the list above is/are determined.

When determining the FLS response, the cumulative number of stress cycles should be used to estimate the fatigue lives of steel components (see 10.6). Clause 10 is specifically aimed at short-term operations where fatigue is typically not a consideration. However, fatigue response can be important for long-term applications of a jack-up (see Clause 11).

A.10.3 Types of analyses and associated methods

The extreme storm ULS response can be determined either by a two-stage deterministic storm analysis procedure using a quasi-static analysis that includes an inertial loadset (see A.10.5.2) or by a more detailed fully integrated (random) dynamic analysis procedure that uses a stochastic storm analysis (see A.10.5.3).

Table 10.3-1 gives a list of some of the references used in an extreme storm response analysis. A common approach can be to start with a relatively simple analysis and to increase the level of complexity if the simple method shows the jack-up is unsuitable for the site.

Table A.10.3-1 — Cross references for extreme storm responses

Topic	Reference location	Comments and additional references
Metocean action calculation procedure	Table A.7.3-1	A.7 discusses actions, but Table A.7.3-1 gives an overview of the calculation procedure and gives references to the required input data, methods of calculating actions, and action factors.
Structural model	A.8	Table A.8.2-1 discusses the levels of detail in different structural models, and the information that can be obtained from them. A.8.3 to A.8.5 discuss modelling of the legs (including some simplified methods for calculating equivalent leg stiffness properties), the hull, and the leg to hull connection, respectively. A.8.7 discusses mass modelling.
Action factors	8.8	Action factors are given for both the two-stage, and one-stage stochastic storm analysis.
Application of actions	A.8.8	Wind and wave/current actions are determined through use of A.7.3. A.8.8 discusses application of actions, including functional actions, hull sagging, metocean actions, and inertial actions. Additional load cases that should be considered when $(T_n/T_p) > 0{,}9$, are given in A.10.5.2.2.3.
Large displacement effects	A.8.8.6	Different modelling techniques are discussed, including large displacement methods, geometric stiffness methods and negative springs.
Conductor actions	A.8.8.7	—
Damping	A.10.4.3	Table A.10.4-1 gives recommended explicit damping levels. A.7.3.3.2 describes relative velocity hydrodynamic damping and Formula (A7.3-15) gives the specific limits for when relative velocity formulation may be used. A.10.4.3.3 describes the hysteretic foundation damping that may be used in certain cases.
Two-stage deterministic storm analysis	A.10.5.2	In this method, a DAF is calculated and used to develop an inertial loadset that is combined with the maximum quasi-static wave action. The DAF can be from an SDOF analysis (A.10.5.2.2.2) or a random dynamic analysis (A.10.5.2.2.3). Figure A.10.4-2 gives an overview of a two-stage approach incorporating foundation fixity, which is normally included in the analysis. A.10.4.4.1.2 gives foundation modelling for a two-stage analysis.
SDOF analysis	A.10.5.2.2.2	This method is very simple and often used for a first pass assessment, but it has a limited range of applicability and, while normally conservative, can underestimate the DAF.

Topic	Reference location	Comments and additional references
Random dynamic analysis	A.10.5.2.2.3	Commonly used to develop the dynamic response and then the DAF in a two-stage analysis Sets out the metocean and inertial loadset components of the basic load case that should be assessed for all values of (T_n/T_p), and the extra load cases that should be considered when $(T_n/T_p) > 0,9$ Table A.7.3-3 gives specific recommendations on qualifying storm simulations. A.6.4.2.3 gives information on wave spreading using either three-dimensional analysis or a kinematics reduction factor. ISO 19901-1:2015, 8.4.4 and A.8.4.3, give information on the intrinsic and apparent wave periods, and the methodology for modifying the wave spectrum from intrinsic to apparent. A.10.5.3.3 gives additional details on all random wave dynamic analyses, regardless of whether it is for a one-stage or two-stage assessment. A.10.5.3.4 gives information on determining the MPME response, which is the result of the random analysis. Table A.10.5-1 gives recommendations for calculating the MPME and on the storm duration to use in the simulations.
Stochastic storm analysis	A.10.5.3	In this method, the MPMEs of the responses of interest (e.g. member utilizations) are determined directly in a one-stage analysis, although multiple one-stage dynamic analyses can be required (10.5.3). DAFs are not specifically developed. A.10.5.3.2 describes the determination and application of partial factors to the metocean parameters, as required in 10.5.3. Figure A.10.5-4 shows the analysis procedure for a one-stage stochastic storm analysis including foundation fixity. A.10.5.3.4 describes the determination of MPME responses.
Leg inclination	A.10.5.4	The effect of leg inclination is included in the structural code checks, but not in the global response analysis.

A.10.4 Common parameters

A.10.4.1 General

The ULS response can be calculated either by using a quasi-static analysis procedure including an inertial loadset or by using a more detailed (random) dynamic analysis procedure.

Clause 8 and A.8 identify the factors that affect the structural stiffness of the jack-up and discuss the structural stiffness modelling at various levels of complexity. The actions are discussed in Clause 7 and A.7.

The magnitude of the dynamic response is affected by the following:

a) the dynamic characteristics (natural periods) of the structural system formed by the jack-up on its foundation;

b) the characteristics of the excitation. For metocean excitation at sites with high current, there can be significant contributions from higher order harmonics in addition to those normally associated with quadratic drag terms and free surface effects.

The factors that affect these two characteristics are discussed in A.10.4.2 to A.10.4.5

A.10.4.2 Natural periods and affecting factors

A.10.4.2.1 General

The natural period of the jack-up on its foundation in the fundamental (or first) mode of vibration is an important indicator of the degree of dynamic response to be expected. The first and second vibrational modes are normally the surge and sway modes. The natural periods of these vibrational modes are usually close together; which of the two is the higher depends on which direction is less stiff. Where the natural or wave period varies with heading, care should be taken that the periods used are applicable to the direction being considered in the analysis. The third vibrational mode is normally a torsional mode, the three-dimensional effects of which can be important, in particular for headings where the legs and, hence, wave actions are not symmetric about the direction of wave propagation.

The natural period is dictated by the characteristics of the structural system, which are governed by the overall (global) structural stiffness, the mass and mass distribution, and the damping.

The undamped natural period is determined from Formula (A.10.4-1):

$$T_n = 2\pi\sqrt{(M / K)} \quad (A.10.4\text{-}1)$$

where

T_n is the first natural period of surge or sway motion of the jack-up;

M is the effective system mass;

K is the effective system stiffness.

ISO/TR 19905-2 contains a manual method for calculating the natural period. The method is not recommended for use in analyses but is useful for demonstrating some of the factors that affect the natural period of a jack-up.

A.10.4.2.2 Stiffness

The jack-up on its foundation represents a multi degree-of-freedom system. If available, a finite element structural model, containing the mass and stiffness properties of the jack-up should be used to obtain the various natural periods and mode shapes. Structural modelling at various levels of complexity is discussed in A.8 and should consider stiffness contributions from the following:

a) bending deformation of the legs;

b) shear deformation of the legs;

c) axial deformation of the legs;

d) hull bending deformation;

e) horizontal vertical and rotational leg-to-hull connection stiffness;

f) horizontal, vertical and rotational foundation stiffness;

g) second order P-Δ due to lateral displacement of the hull;

The model can contain a number of non-linear elements, notably the leg-to-hull connections and the spudcan-foundation interfaces.

The system stiffness for the fundamental modes can be estimated from an idealized single degree-of-freedom system as described in ISO/TR 19905-2. The method is not recommended for use in analyses but is useful for demonstrating some of the factors that affect the natural period of a jack-up.

A.10.4.2.3 Mass

No guidance is offered.

A.10.4.2.4 Variability in natural period

No guidance is offered.

A.10.4.2.5 Cancellation and reinforcement

A.10.4.2.5.1 General

If the legs of the jack-up were lumped together at one position, waves passing through would cause each leg to have the same applied force history and the base shear transfer function (base shear versus wave period) would be a relatively smooth function. Assuming the leg kinematic parameters are axisymmetric, this transfer function would be the same for all wave headings. As the legs are moved apart, at an instant in time the wave position relative to each leg is different for each wave period. Since each leg is at a different phase for each wave period, the amplitude of the base shear transfer function at every period is bounded by the value with all legs together. Essentially, there is some force cancellation for almost all periods (smaller amplitudes than all legs together). Since the spacing between the legs changes by approach direction, different wave headings also result in different base shear transfer functions, even if the kinematic properties are still axisymmetric.

Figure A.10.4-1 shows cancellation and reinforcement periods. It can be used for a first evaluation of the position of the calculated natural period(s) relative to the cancellation and reinforcement points in the global loading. These can be characterized by the total horizontal wave loading or by the overturning moment; cancellation and reinforcement of points for these can occur at slightly different wave periods.

The assessor should aim to maximize the overall jack-up responses and not just, for example, the DAF.

The DAF calculated through the SDOF is independent of cancellation and reinforcement.

A.10.4.2.5.2 Quasi-static deterministic waves

Care should be taken to avoid cancellation in the quasi-static deterministic wave actions. This is not normally an issue; rarely is the extreme storm wave period close to a cancellation period, but if it is, a range of wave periods should be investigated (see A.6.4.2.9 and A.6.4.2.3).

A.10.4.2.5.3 Stochastic dynamic wave response

The natural period(s) used in the dynamic analysis should be selected such that a realistic but conservative value of the dynamic response is obtained for the particular application envisaged. Care should be taken to ensure that the response is maximized, not just the dynamic amplification, since it is possible to have a large DAF combined with low metocean excitation, due to cancellation, leading to low combined response. When the DAF is determined through a stochastic analysis, care should be taken to minimize cancellation (see also A.7.3.3.3.3) as this can result in significant underestimation of the DAF. In a two-stage stochastic analysis, the DAF is determined as the ratio of the responses of two models (see A.10.5.2.2.3): one that includes and one that excludes dynamic effects. A significant percentage of the dynamic effect is due to excitation of the natural period of the jack-up by that component of the wave trace having that same period. If there is cancellation at that period, there is little excitation, so the calculated DAF is unrealistically small.

Care should also be taken when there is significant current velocity as this can lead to slightly different cancellation effects. When combining current with a cyclic Morison wave loading, the drag term causes a harmonic excitation at half the wave period. This second harmonic can result in significant dynamic excitation, especially when the current is large and the period of the second harmonic is the same as the natural period. If cancellation of the second harmonic actions occurs, the DAF can be significantly underestimated.

In order to prevent cancellation resulting in potential underestimation of the DAF, the range of possible natural period(s) should be bracketed and compared with the relevant cancellation points in the global wave loading and the second harmonic of the wave period. When the natural period occurs at a cancellation point in the transfer functions, the mass or stiffness should be adjusted in a logical manner to move the natural period away from the cancellation point. This generally ensures that the dynamic response is maximized within reasonable limits.

The definitive selection of natural period(s) should be based on the shape of the global horizontal wave loading (base shear) and overturning moment transfer functions for the case under consideration.

If the analysis is for pinned spudcans with maximum hull mass, then the adjustment should be made by reducing the hull mass (within the normal range) and/or by introducing a degree of rotational fixity at the seabed.

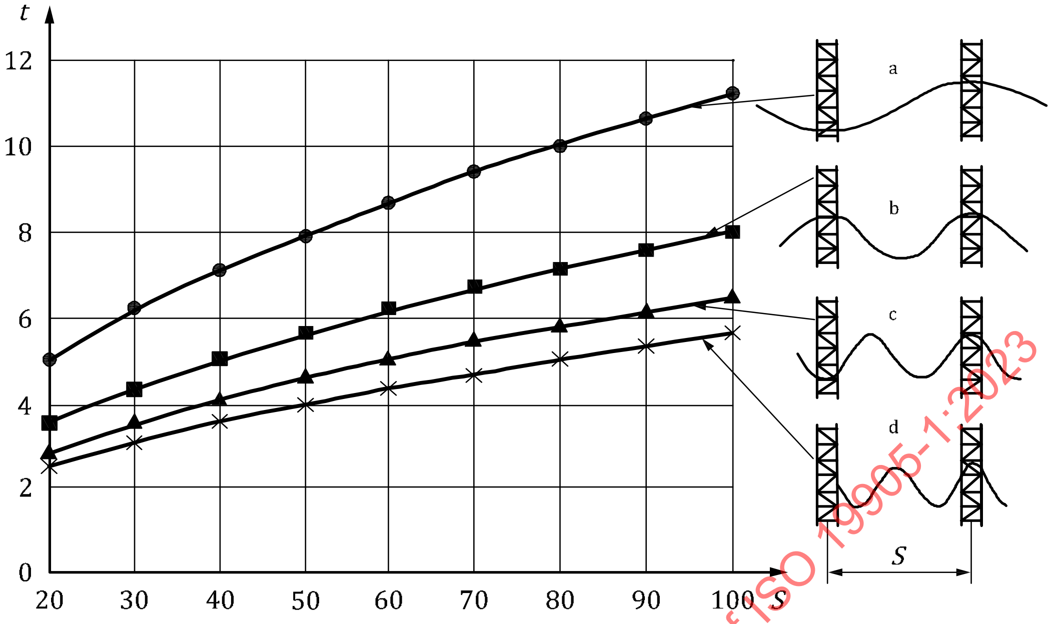
If the analysis is for a case with a degree of spudcan moment fixity, then the adjustment can most logically be made by varying the degree of rotational fixity at the seabed.

Alternatively, when the metocean data is omni-directional, the effects of wave spreading can be used to reduce the effects of cancellation by carrying out the dynamic analysis for a single wave heading along an axis which is neither parallel nor normal to a line through two adjacent leg centres. Thus, for a 3-legged jack-up with equilateral leg positions and a single bow leg, suitable analysis headings can be with the environment approaching from approximately 15° or 45° off the bow. The DAFs should be determined for one, or both, of these headings. The DAFs (or more conservative DAFs) can then be applied to the final quasi-static analysis for all headings.

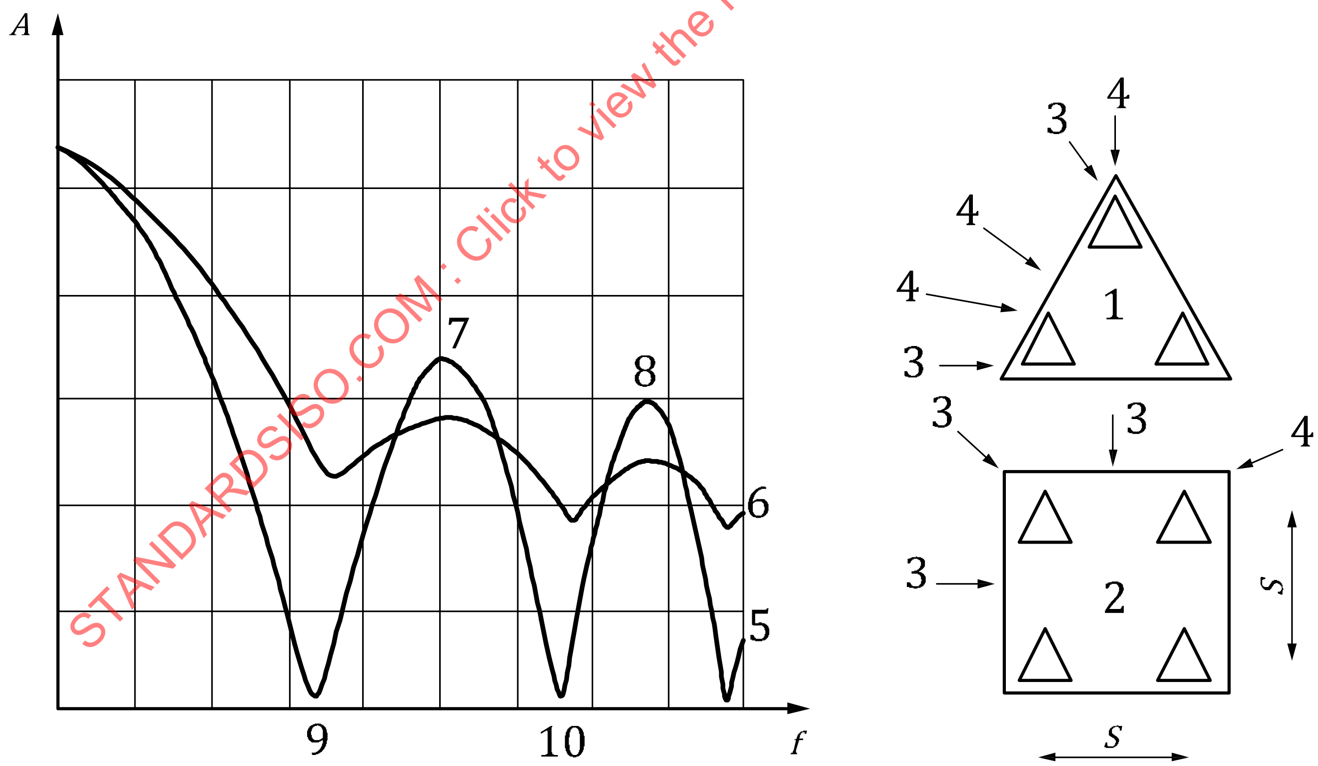
In a one-stage stochastic analysis, similar care should be taken to avoid cancellation effects at both the natural period and at the predominant wave spectral energy.

Figure A.10.4-1 presents the periods at which first and second cancellations and reinforcements occur in the total wave actions. It is valid for the main wave directions of 3- and 4-legged jack-ups in water depths exceeding 30 m. The potential for increased response due to short-crested waves should be considered (see A.7.3.3.3.3).

Dimensions in metres

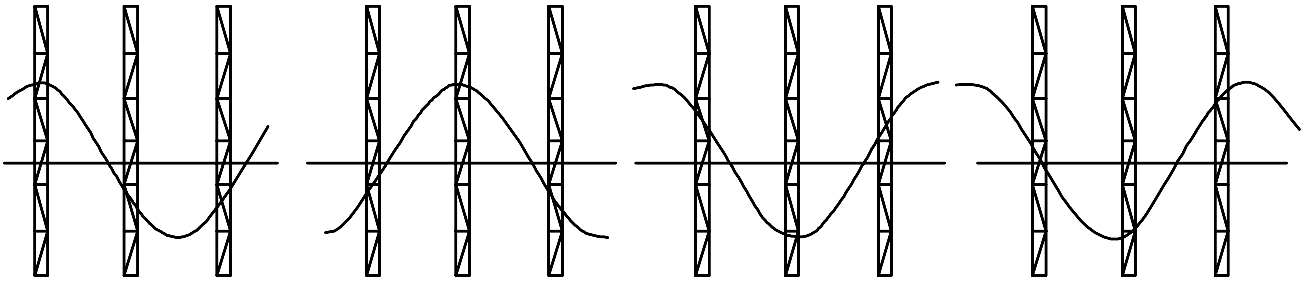


a) Sample of wave period in relation to wave force cancellation and reinforcement at all phase angles, including diagrammatic arrangement of jack-up legs with wave length



b) Horizontal action on jack-up versus wave frequency showing reinforcement and cancellation

Figure A.10.4-1 — Periods for wave force cancellation and reinforcement as a function of leg spacing
(1 of 2)



c) Diagrammatic arrangement of legs on 3-legged jack-up in beam seas that can result in complete horizontal wave action cancellation at all wave phase angles

Key

1 3 legged jack-up
2 4 legged jack-up
3 wave direction versus leg locations associated with wave action curve 5
4 wave direction versus leg locations associated with wave action curve 6
5 indicative curve of wave action on jack-up versus frequency due waves in directions 3
6 indicative curve of wave action on jack-up versus frequency due waves in directions 4
7 first reinforcement point
8 second reinforcement point
9 first cancellation point
10 second cancellation point

A static wave action on jack-up
f wave frequency
t wave period
S jack-up leg spacing

[a] First wave force cancellation over all wave phase angles; water depth ≥ 50 m.

[b] First wave force reinforcement over all wave phase angles; water depth ≥ 30 m.

[c] Second wave force cancellation over all wave phase angles; water depth ≥ 30 m.

[d] Second wave force reinforcement over all wave phase angles; water depth ≥ 30 m.

NOTE 1 Figure A.10.4-1 a) has been drawn for effectively deep water cases only. The reduced wave length in shallow water results in slightly longer wave periods producing first cancellation.

NOTE 2 On a 4-legged jack-up, it is possible to get complete cancellation of the horizontal actions at certain wave lengths (e.g. in a wave of specific length that results in two legs at the wave crest and two at the wave trough, as shown by line 'a' in Figure A.10.4-1 a). It is not possible to get complete cancellation of the horizontal actions on a 3-legged jack-up oriented with two legs parallel to the wave crest. There is partial cancellation in waves that result in one leg at a trough when two legs are at a crest, as shown by line 6 of Figure A.10.4-1 b), but there is not sufficient cancellation in any wave length to result in line 5. It can be possible to get complete cancellation on a 3-legged jack-up oriented with any two legs parallel to the direction of wave propagation, as shown in Figure A.10.4-1 c), but it is not for precisely the wave periods given in Figure A.10.4-1 a).

Figure A.10.4-1 — Periods for wave force cancellation and reinforcement as a function of leg spacing
(2 of 2)

A.10.4.3 Damping

A.10.4.3.1 General

The main components of system damping are foundation, hydrodynamic and structural damping. Each of these can be modelled either linearly or non-linearly and can be calculated as part of the analysis or input as a percentage of critical damping (see Table A.10.4-1).

Structural damping is normally modelled linearly and input as a percentage of critical damping, however there are non-linear components (e.g. gaps in guides, pinion backlash).

Hydrodynamic damping is mainly due to fluid-structure relative velocity effects (see A.7.3.3.2); alternatively, a percentage of critical damping can be applied.

Foundation damping comprises three components: small strain material, hysteretic and radiation damping. The small-strain soil material damping is typically small. At larger strains, amplitude-dependent hysteretic damping can also occur. Where a non-linear foundation model is adopted for dynamic response analysis, the hysteretic foundation damping and soil stiffness reduction are accounted for directly. Where linearized soil stiffness is used in a time domain analysis, hysteretic damping should not be included.

A.10.4.3.2 Linear system damping

Where the model relies on damping defined as a percentage of critical, the total linear system damping should not exceed 7 % without credible, applicable justification. Lower values can be appropriate for fatigue analyses and lower sea states. Care should be taken to avoid the duplication of damping components when explicit and implicit representations are used simultaneously in the analyses. Table A.10.4-1 summarizes typical upper bounds when using percentages of critical damping.

Table A.10.4-1 — Recommended explicit damping from various sources

Damping source	Global linear damping not to exceed (% of critical damping)
Structure, holding system, etc.	2
Small strain foundation	2[a]
Hydrodynamic	3 or 0[b]

[a] The small-strain soil material damping is typically small; in the absence of specific data, 2 % is considered to be a reasonable estimate.

[b] In cases where the relative velocity formulation is used [$\alpha = 1$ in Formula (A.7.3-15)], the hydrodynamic damping is accounted for directly and should not be included as a percentage of critical damping.

A.10.4.3.3 Hysteretic damping

Foundation hysteretic damping can, in certain situations, increase the 2 % small-strain foundation damping given in Table A.10.4-1 and is discussed further in ISO/TR 19905-2.

A.10.4.3.4 Vertical radiation damping in earthquake analysis

In earthquake analyses, the foundation radiation damping from wave propagation can be included for vertical motion of the spudcan in addition to other foundation damping. Radiation damping should not normally be used in extreme storm or fatigue jack-up assessments. Radiation damping effects are implicitly included when the dynamic foundation analysis is performed using a continuum finite element analysis with a model that can accurately capture the effects of wave propagation in the foundation soils. Additional information on radiation damping is given in ISO/TR 19905-2. In simpler analyses, the vertical foundation radiation damping can be estimated from the work of Lysmer and Richart (1966)[130], as given in Formula (A.10.4-2):

$$C_{rd} = R\,[0{,}85\,B^2/(1-\nu)]\,\sqrt{(G_o \rho_s)} \qquad \text{(A.10.4-2)}$$

where

C_{rd} is the radiation damping coefficient of a dashpot (force per unit velocity);

R is a reduction factor applied to avoid unconservatism, which should normally be taken as 0,5;

B is the effective spudcan diameter at uppermost part of bearing area in contact with the soil;

ν is Poisson's ratio (of the foundation soil);

G_o is the shear modulus of the foundation soil [for clay, $G_o = G_{max}$, the maximum value of the shear modulus, that occurs at small strain (see A.9.3.4.3); for sand, $G_o = G_{max}$, the initial small-strain shear modulus (see A.9.3.4.4)];

ρ_s is the total, saturated, (mass) density of the foundation soil.

In non-linear dynamic analyses, or in linear time domain dynamic analyses using direct integration, Formula (A.10.4-2) can be used directly to establish the damping coefficients for the foundation dashpots.

In linear modal dynamic analyses, the additional contribution of vertical radiation damping to the linear damping ratio for the vertical mode only can be calculated as given in Formula (A.10.4-3):

$$\zeta_{rd} = R\,0{,}213\,N_s\,B\,\omega_n\,\sqrt{(\rho_s/G_o)} \qquad \text{(A.10.4-3)}$$

where

ζ_{rd} is the radiation modal damping ratio to account for spudcan vertical motion;

N_s is the number of spudcans;

ω_n is the angular natural frequency of the vertical mode, expressed in radians per second.

NOTE 1 The suggested value of 0,5 for R is a reduction on the amount of radiation damping and is comparable with values used in other industries. The reduction is intended to account for the frequency dependence and spatial variance (e.g. stratification) in soil conditions below the spudcan.

NOTE 2 Formula (A.10.4-2) is obtained by combining the definition of the damping coefficient, C, with the damping ratio of Formula (A.10.4-3) and the corresponding formula for stiffness given by Lysmer and Richart (1966)[130].

NOTE 3 Radiation damping increases with increasing excitation frequency. Radiation damping levels from ocean wave excitation are expected to be less than 1 %, whereas for earthquake actions, radiation damping ratios can be large (>10 %). Radiation damping values this large can have significant effects on dynamic response.

A.10.4.4 Foundations

A.10.4.4.1 Foundations for extreme storm assessment

A.10.4.4.1.1 General

A.10.4.4.1 describes the analysis of the structure and the foundation evaluation which can be performed in two different ways:

- option 1: deterministic two-stage approach;
- option 2: stochastic one-stage approach.

A.10.4.4.1.2 Option 1 — Deterministic two-stage approach

Figure A.10.4-2 illustrates the procedure schematically.

In this approach, the dynamic response of the structure is evaluated based on either a simple linear analysis or a more complex elasto-plastic analysis in order to determine an inertial loadset. The dynamic analysis can include linearized springs. Typically, the initial linearized rotational stiffness for the dynamic analysis can be taken as 80 % of the value determined from A.9.3.4.1. This simplified approach does not capture the temporary reductions in stiffness that occur during plasticity events (generally with detrimental effects), but also does not capture the increased damping associated with these events (with beneficial effects).

The foundation and structural assessment is next performed using a quasi-static, iterative analysis technique, for which the dynamic actions have already been determined. This quasi-static analysis can be accomplished by means of either an elasto-plastic foundation model or by a simplified application of the full plasticity analysis as described below. This simple approach is used to apply moments on the spudcan by inclusion of a simple linear rotational spring. The moments thus applied are limited to a capacity based on the yield interaction relationship between the gross vertical force (F_V), the horizontal force (F_H) and the moment (F_M) acting on the spudcan.

This simple procedure is described in the following steps (see the right hand side of Figure A.10.4-2).

a) Include vertical, horizontal and (initial) rotational stiffnesses (using linear springs, see A.9.3.4.1) in the analytical model and apply the factored functional and factored metocean actions together with the associated and separately calculated inertial loadset from a linearized dynamic analysis, to determine the resulting forces F_H, F_V and the moment F_M on each spudcan.

b) Calculate the value of the yield interaction function (see A.9.3.3) using the resulting forces on each spudcan. If the value is zero, the force combination falls on the yield surface; for values greater than zero, it is outside; and for values smaller than zero, it is inside the yield surface.

c) If a force combination initially falls within the yield surface, the rotational stiffness should be further checked to satisfy the reduced stiffness conditions in A.9.3.4.2.

d) If the force combination initially falls outside the yield surface, the rotational stiffness should be arbitrarily reduced and the analysis should be repeated until the force combination at each spudcan lies essentially on the yield surface. If at that point the moment is reduced to zero and the force combination is still outside the yield surface, then a bearing failure (either vertical or horizontal) is indicated.

e) Additional penetration due to a bearing failure can result in increased foundation capacity, which, in turn, expands the yield interaction surface. See A.9.3.3.5 and A.9.3.3.6 for guidance on expansion of yield interaction surface and A.9.3.6.6 for guidance on the displacement check.

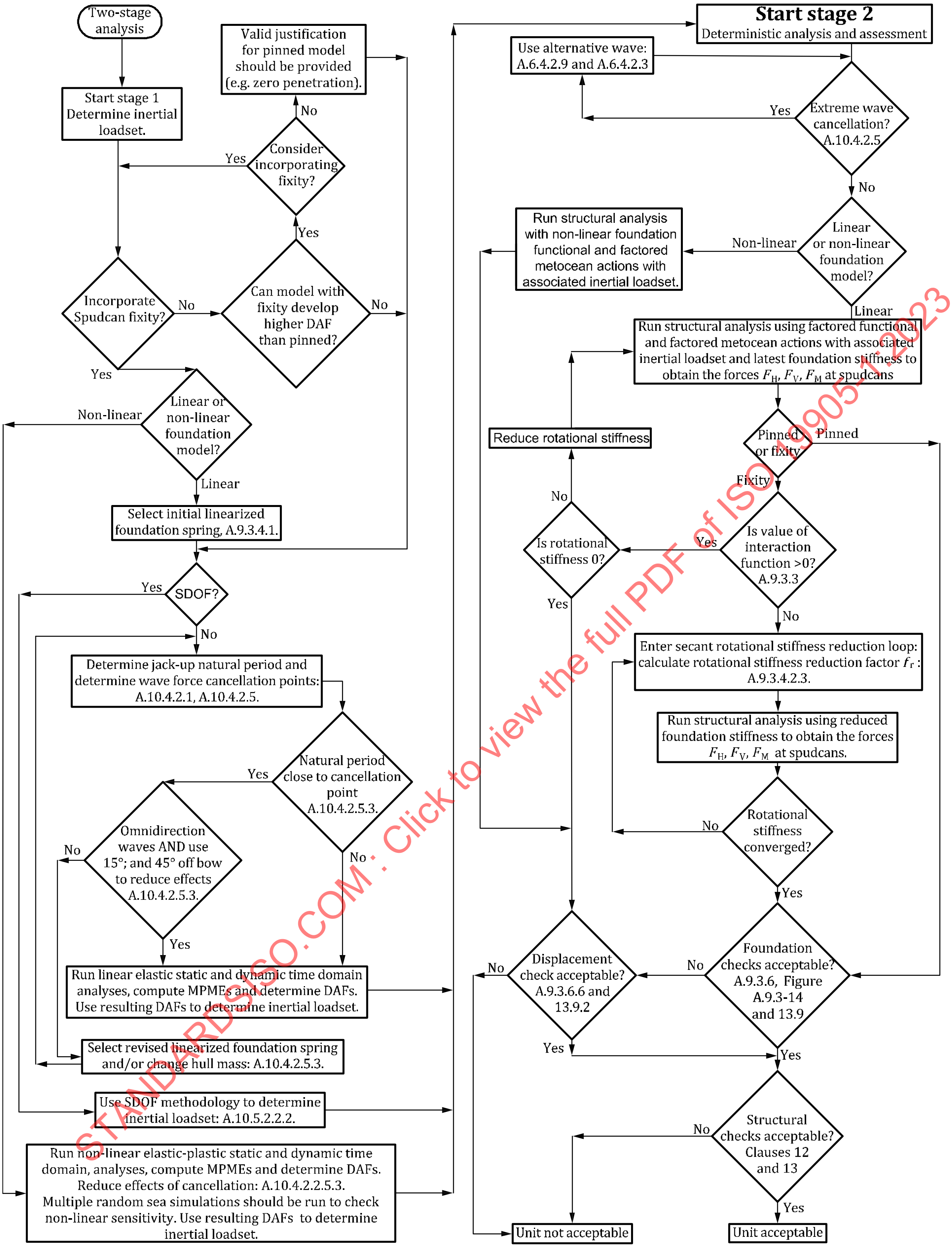


Figure A.10.4-2 — Analysis procedure for two-stage assessment with foundation fixity — Option 1

A.10.4.4.1.3 Option 2 — Stochastic one-stage approach

Figure A.10.5-4 illustrates the procedure schematically.

In this approach, the dynamic structural analysis and assessment is performed using one model. A fully detailed, non-linear time domain analysis is performed taking into account the elasto-plastic behaviour of the foundation.

The effects of the foundation fixity on the dynamic response and on the foundation reactions are simultaneously considered. This approach is more complete and often requires a complex incremental and iterative calculation procedure. The following outline procedure can be used.

a) Use a time domain random dynamic analysis to determine structural response and foundation forces at each time step.

b) Determine the foundation behaviour using a non-linear elasto-plastic model, such that at each time step the plastic and elastic portions of the behaviour are captured. If desired, this model can include hysteresis. This is likely to require an iterative procedure.

c) As the dynamic response is influenced by the time history of the actions, a number of random dynamic analyses should be performed for differing input wave histories, and the MPMEs determined from a procedure described in A.10.5.3.4.

If, due to wave force cancellation effects, small changes in foundation stiffness result in significant changes in the response, the foundation stiffness should be selected with care to maximize the response (see A.10.4.2).

A.10.4.4.2 Foundations for earthquake assessment

For the simple screening assessment, the foundation should be modelled with a high representative value of G_0 from Clause 9, without degradation but with appropriate rate adjustments.

For more detailed assessments, a fully non-linear coupled yield interaction model or a continuum model should be used with degradation effects.

A.10.4.5 Storm excitation

Currents change slowly compared with the natural periods at which jack-ups oscillate and can be considered to be a steady phenomenon. Variations in wind velocity cover a wide range of periods, but the main wind energy is associated with periods that are considerably longer than the natural periods of jack-up oscillations. Therefore, the wind can generally also be represented as a steady flow of air. The periods of waves typically lie between 3 s and 20 s. Since natural periods of jack-up in typical applications fall within this range, the primary source of dynamic excitation is from waves.

Sea waves are not regular but random in nature, with a more predominant periodicity when a swell is present. This has important implications that should be considered for both the dynamic excitation and the resulting dynamic response.

A.10.5 Storm analysis

A.10.5.1 General

No guidance is offered.

A.10.5.2 Two-stage deterministic storm analysis

A.10.5.2.1 General

In the first stage, an inertial loadset is determined from a dynamic amplification factor using either a single degree-of-freedom analogy ($K_{\text{DAF,SDOF}}$), see A.10.5.2.2, or a random wave time domain random dynamic analysis ($K_{DAF,RANDOM}$), see A.10.5.2.2.3. In the second stage, the maximum quasi-static wave/current action is determined by stepping the maximum wave through the structure. The maximum wave/current action is then combined with the inertial loadset to determine the responses. The maximum wave is defined in 6.4 and the methodology for calculating the quasi-static wave/current actions is described in 7.3. Load cases and combinations are discussed in 8.8.

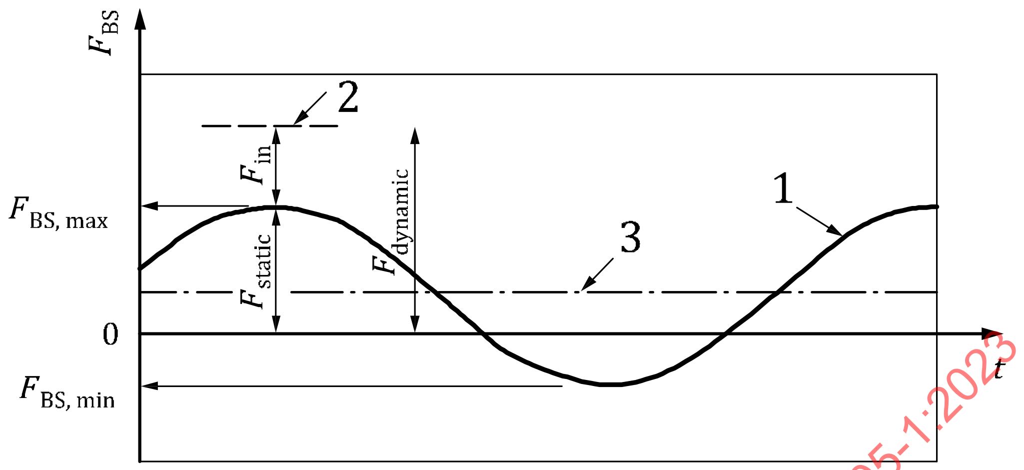
The spudcan-foundation interface can be modelled as described in 9.3.1.

A.10.5.2.2 Dynamic amplification factors (DAFs) and inertial loadsets

A.10.5.2.2.1 General

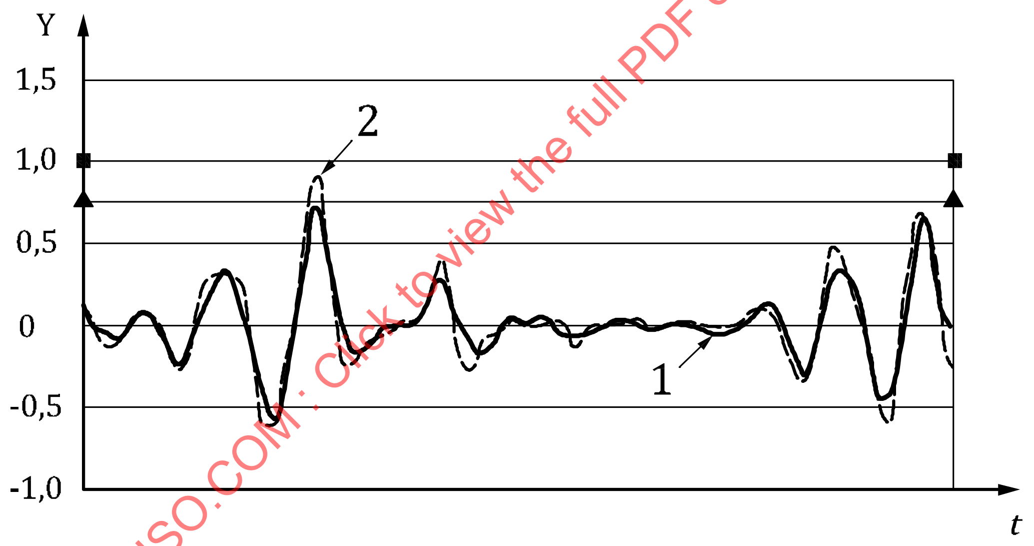
When using a deterministic analysis for calculating the jack-up's responses, the dynamic response is represented by equivalent inertial actions as described in A.8.8.5. The inertial loadset can be derived from the classical SDOF analogy described in A.10.5.2.2.2, or from the more complex random dynamic analysis method discussed in A.10.5.2.2.3; see Figure A.10.5-1. It should be recognized that dynamic amplification is the result of inertial actions that are dominated by the hull mass. Therefore, amplifying the hydrodynamic actions is not a correct physical representation.

NOTE The difference between the height of the applied wave actions and the height of the system centre of mass means that the global response (e.g. base shear, overturning moment, hull deflection) and local response (e.g. member forces, holding system reactions, spudcan reactions) are not equally amplified by the inertial actions.



$F_{in} = (K_{DAF,SDOF} - 1)[A_{BS(t)static}]$
where $A_{BS(t)static}$ is the amplitude of $F_{BS(t)static}$, is equal to $0,5(F_{BS,max} - F_{BS,min})$

a) SDOF



$K_{DAF,RANDOM} = R_{MPME,dynamic}/R_{MPME,static}$
$F_{in} = (K_{DAF,RANDOM} - 1)\ F_{static}$

b) Stochastic/random

Key

t	time	*BS*	base shear
Y	normalized stochastic response	$F_{BS,min}$	minimum static base shear
1	static response	$F_{BS,max}$	maximum static base shear
2	dynamic response	F_{in}	magnitude of inertial loadset
3	mean of static response	$F_{dynamic}$	magnitude of total dynamic loadset
▲	$R_{MPME,static}$ stochastic response	F_{static}	magnitude of total static loadset
■	$R_{MPME,dynamic}$ stochastic response	R_{BS}	base shear response

Figure A.10.5-1 — Dynamic amplification factors

A.10.5.2.2.2 The classical SDOF analogy ($K_{DAF,SDOF}$)

This representation assumes that the jack-up on its foundation can be modelled as an equivalent single degree-of-freedom mass-spring-damper mechanism. The (highest) natural period of the jack-up's vibrational modes can be determined as described in A.10.4.2. The torsional mode and corresponding three-dimensional effects cannot be included in this representation.

The SDOF method is fundamentally empirical because:

— the wave/current action does not occur at the hull;

— the excitation is non-periodic (random) and non-linear.

The method described below generally leads to an approximation of the jack-up's real behaviour that has been calibrated against more rigorous methods. The following cautions are noted when using the SDOF method.

a) If the ratio of the jack-up natural period to the wave excitation period, Ω, is in the range 0,4 to 0,8 and the current velocity is small relative to the wave particle velocities, the SDOF method can give reasonable results, subject to items b) to d) below.

b) The SDOF method does not account for reinforcement, as discussed in A.10.4.2.4, and this can make the method unconservative, particularly when $\Omega > 0{,}5$. When $\Omega > 0{,}5$, there can be significant energy in an irregular sea at the jack-up natural period, and this is not accounted for in the SDOF method because the DAF is not affected by any periodicity other than the excitation at $0{,}9T_p$. This lack of excitation is particularly important when the jack-up natural period is close to a wave reinforcement point. In this case, the resonant response, combined with reinforcement, can result in a significantly higher action than that calculated from the SDOF method. In the calculation of the natural period, a range in foundation fixity should be considered as this variability can shift the jack-up natural period within the base shear transfer function, resulting in different dynamic amplifications.

c) The SDOF method can be unconservative for cases where the current velocity is large relative to the wave particle velocities. If the results of the assessment are close to the acceptance criteria, further detailed analysis is recommended.

d) The SDOF method can be unconservative and should not normally be used in an extreme storm assessment when Ω is greater than 1,0, i.e. when $T_n > 0{,}9T_p$. However, the SDOF analogy may be used when the calculated Ω is greater than 1,0 providing Ω is taken as 1,0.

When using the SDOF method, a minimum value of 1,2 should be taken as the DAF in an extreme storm assessment, regardless of the DAF calculated using the SDOF method.

NOTE The DAF calculated in the SDOF analogy ($K_{DAF,SDOF}$) cannot be meaningfully compared to the DAF determined with a stochastic wave assessment ($K_{DAF,RANDOM}$). This is because the methods for determining the relevant inertial loadsets are different, thus the same value of $K_{DAF,SDOF}$ and $K_{DAF,RANDOM}$ produce different total global responses; see Figure A.10.5-1.

The ratio of (the amplitudes of) dynamic to quasi-static response as a function of frequency (ω) or period (T) steady state, periodic and sinusoidal excitation is calculated by means of the classical dynamic amplification factor ($K_{DAF,SDOF}$) as given in Formula (A.10.5-1):

$$K_{DAF,SDOF} = \frac{1}{\sqrt{(1-\Omega^2)^2 + (2\zeta\Omega)^2}} \geq 1{,}20 \tag{A.10.5-1}$$

where

Ω is the jack-up's natural period (T_n) divided by the wave excitation period (T_w); $\Omega = \frac{T_n}{T_w} \leq 1,0$;

ζ is the damping ratio or fraction of critical damping, $\zeta \leq 0,07$ (see A.10.4.3);

T_w $= 0,9 T_p$;

T_p is the apparent peak wave period (modal or most probable period of the wave spectrum, corrected to account for current velocity; see A.7.3.3.5 and ISO 19901-1:2015, A.8.4.3);

T_n is the natural period as derived in A.10.4.2.1.

The damping parameter, ζ, in this model represents the total of all damping contributions (structural, hydrodynamic and foundation damping). For the evaluation of extreme jack-up responses using the SDOF method, a value not exceeding 0,07 is recommended.

The calculated $K_{DAF,SDOF}$ from the SDOF analogy is used to estimate an inertial loadset, which represents the contribution of dynamics over and above the quasi-static response as illustrated in Figure A.10.5-1 a). The inertial loadset should be determined as given in Formula (A.10.5-2) and applied at the hull centre of gravity in the direction of wave propagation:

$$F_{in} = (K_{DAF,SDOF} - 1)\, F_{BS,Amplitude} \quad \text{(A.10.5-2)}$$

where

F_{in} is the magnitude of the inertial loadset;

$F_{BS,Amplitude}$ is the single amplitude of quasi-static base shear over one wave cycle,

$F_{BS,Amplitude} = [F_{BS,(QS)Max} - F_{BS,(QS)Min}]/2$;

$F_{BS,(QS)Max}$ is the maximum quasi-static wave/current base shear;

$F_{BS,(QS)Min}$ is the minimum quasi-static wave/current base shear.

Formula (A.10.5-2) is part of a calibrated procedure and should not be altered. A more general inertial loadset procedure, using the results from random dynamic analysis, is described in A.10.5.2.2.3.

A.10.5.2.2.3 Inertial loadset based on random dynamic analysis ($K_{DAF,RANDOM}$)

In the time domain random dynamic analysis procedure, two DAFs are calculated, one for the BS and one for the overturning moment (OTM). The inertial loadset, F_{in}, is calculated from these DAFs. The BS and OTM DAFs are the ratios of the MPME of the dynamic BS/OTM to the MPME of the static BS/OTM ($R_{MPME,dynamic}/R_{MPME,static}$), see Figure A.10.5-1 b), determined from corresponding dynamic and quasi-static time domain analyses for random-wave excitation according to the recommendations of the stochastic storm analysis in A.10.5.3. The MPME is defined in Table A.10.5-1.

Damping effects, including relative velocity effects, should not be included in the quasi-static (zero mass) analysis.

P-Δ effects should be included in both the quasi-static (zero mass) and the dynamic analyses. When P-Δ effects are included using negative springs, the same springs should be used in both analyses, although when calculating the BS DAF the shear force induced by the negative spring should be excluded. When the P-Δ effects are developed from gravity actions, the effects of vertical gravity loads should be modelled in the zero-mass analysis, i.e. weight is included even though there is no mass.

The inertial loadset, F_{in}, normally should be such that it increases both the BS and OTM from the deterministic quasi-static analysis by the same ratios as those determined between the random quasi-static (zero mass) analysis and the random dynamic analysis. In such cases, the structural model (used for dynamic analysis) may be simplified and it is not necessary that it contain all the structural details, but should nevertheless be a multi degree-of-freedom model. See A.8.8.5 for guidance on applying an inertial loadset to the model that matches both dynamic BS and OTM.

Caution should be exercised when the wave period approaches resonance and additional load cases should be considered when (T_n/T_p) is greater than 0,9. These extra load cases account for the changing phase between the forcing action and the inertial action as (T_n/T_p) approaches and exceeds 1,0 (see Figure A.10.5-3 and NOTE 1). The basic load case is the inertial loadset applied in phase with, and to increase the response to, the metocean actions, Formula (A.10.5-4). This load case is required for all ratios of (T_n/T_p). Three additional load cases, Formulae (A.10.5-5) to (A.10.5-7), should be considered when (T_n/T_p) is greater than 0,9. Four sample load cases are shown diagrammatically in Figure A.10.5-3. In each case, the inertial loadset should be applied to the structure as described with A.8.8.5, using the same directional pair of $K_{DAF,RANDOM}$ values calculated for base shear and overturning moment.

NOTE 1 Figure A.10.5-3 shows the phase between the forcing action and the inertial action for an SDOF system for varying values of T_n/T_p and represents the underlying reason the extra load cases are assessed in a two stage deterministic analysis when T_n/T_p is greater than 0,9. As the value of T_n/T_p increases beyond 0,9 the phase between the exciting action and the inertial action changes from being approximately in phase for low values of T_n/T_p, through being 90° out of phase when $T_n/T_p = 1{,}0$ to being approximately 180° out of phase when T_n/T_p is greater than 1,2. While Figure A.10.5-3 is drawn for an SDOF system, a similar phasing analogy can be made in a random dynamic analysis, albeit without the same degree of fine definition. It is because the phasing is not so well defined in a random sea state that the extra cases are specified above when T_n/T_p is greater than 0,9.

The total base shear and overturning moment is the same in the first three load cases. Formulae (A.10.5-4 to A.10.5-6) provide a match to the base shear but it is still necessary to correct the overturning moment. Both the base shear and overturning moment can be different in the fourth case: Formula (A.10.5-7); see NOTES 5 and 6.

The base shear inertial loadsets are calculated as given in Formula (A.10.5-3):

$$F_{in,PHASE(a)} = K_{DAF,RANDOM} F_{STATIC} - F_{STATIC,PHASE(a)} \quad (A.10.5\text{-}3)$$

and are applied in load cases as given in Formulae (A.10.5-4) to (A.10.5-7):

$$[E_e + \gamma_{f,D} D_e]_{(0)} = F_{WIND} + F_{STATIC} + \gamma_{f,D} F_{in,PHASE(0)} \quad (A.10.5\text{-}4)$$

$$[E_e + \gamma_{f,D} D_e]_{(90)} = F_{WIND} + \gamma_{f,D} F_{in,PHASE(90)} \quad (A.10.5\text{-}5)$$

$$[E_e + \gamma_{f,D} D_e]_{(180)} = F_{WIND} + F_{STATIC.UP} + \gamma_{f,D} F_{in,PHASE(180)} \quad (A.10.5\text{-}6)$$

$$[E_e + \gamma_{f,D} D_e]_{(-180)} = F_{WIND} + F_{STATIC} - \gamma_{f,D} F_{in,PHASE(-180)} \quad (A.10.5\text{-}7)$$

where

$[E_e + \gamma_{f,D} D_e]_{(a)}$ is the combined metocean actions and inertial actions for use as $(E_e + \gamma_{f,D} D_e)$ in Formula (8.8-1);

(a) is a subscript representing the notional phasing of the four different load cases given in Formulae (A.10.5-4) to (A.10.5-7) in which (a) is (0), (90), (180), and (−180), respectively (see NOTE 4);

F_{STATIC} is the deterministic quasi-static wave/current loadset in the direction of the MPME values;

F_{WIND} is the wind loadset;

$F_{STATIC,PHASE(a)}$ is the deterministic quasi-static wave/current loadset for the relevant load case:

it is equal to F_{STATIC} for the normal $PHASE_{(0)}$ case when used to calculate $F_{in,PHASE(0)}$ in Formula (A.10.5-4), which represents the normal case with inertia down-wind and crest wave loading [see Figure A.10.5-2 a)];

when $T_n/T_p > 0{,}9$:

it is equal to 0,0 for the $PHASE_{(90)}$ case when used to calculate $F_{in,PHASE(90)}$ in Formula (A.10.5-5), which represents the inertia only load case [see Figure A.10.5-2 b)];

it is equal to $F_{STATIC.UP}$ for the $PHASE_{(180)}$ case when used to calculate $F_{in,PHASE(180)}$ in Formula (A.10.5-6), which represents the case with inertia down-wind and trough wave loading [see Figure A.10.5-2 c)];

it is equal to $F_{STATIC.UP}$ for the $PHASE_{(-180)}$ case when used to calculate $F_{in,PHASE(-180)}$ in Formula (A.10.5-7), which represents the case with inertia up-wind and crest wave loading [see Figure A.10.5-2 d)];

$F_{STATIC.UP}$ is the deterministic quasi-static wave/current loadset in the up-wind direction (i.e. maximum upwind loadset, which is normally in the opposite direction to the wind action).

Formulae (A.10.5-4) to (A.10.5-7) represent the metocean and dynamic components, E_e and D_e, in Formula (8.8-1). The gravity components G_F and G_V should also be included when developing the complete assessment load case F_d in Formula (8.8-1). The response analysis should include P-Δ and hull sagging effects and the effects of leg inclination should be taken into account (see 7.8).

NOTE 2 It is relatively unusual to undertake a jack-up assessment where T_n/T_p is greater than 1,0 but such situations do exist, e.g. in relatively benign conditions in deep water and with low spudcan fixity. Experience has shown that in some cases the introduction of additional spudcan fixity reduces the natural period to below the wave period and this action results in an increased DAF.

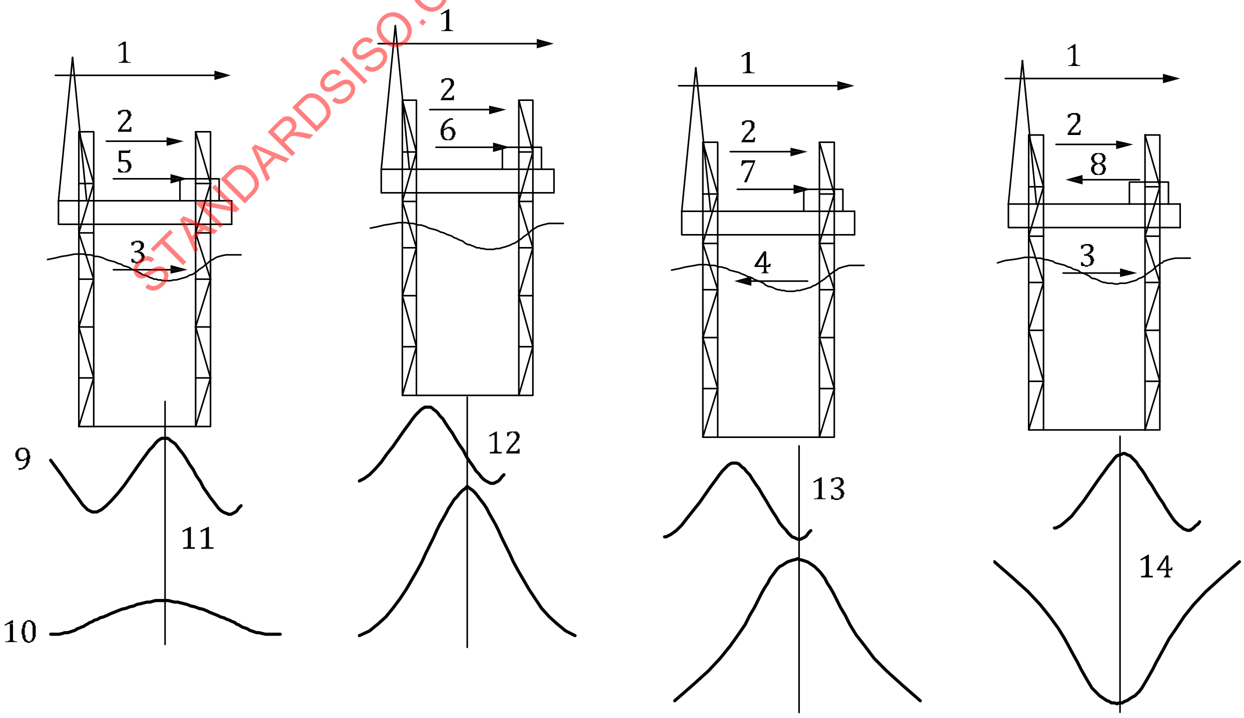
NOTE 3 Formula (A.10.5-3) is a scalar formula. It is used to determine the magnitude of the inertial loadset, but has no associated point of action or direction. Formulae (A.10.5-4) to (A.10.5-7) are vector formulae in which, for example, the inertial loadset is applied in the relevant direction and at the relevant elevation above the seabed.

NOTE 4 The subscript (a) is not the actual phase, but a notional, or indicative, phase taken from the SDOF analogy, similar to that given in Figure A.10.5-3, (in the full knowledge that the assessment is using a multi-degree of freedom model and loading). For example, $PHASE_{(0)}$ is not necessarily at the wave crest. It is simply used to represent the phase when the inertial loading is in-phase with the maximum down-wind wave/current action. Likewise $PHASE_{(90)}$ is used to represent the phase when the wave/current action is zero. $PHASE_{(180)}$ is used to represent the case when the inertia and direct wave/current actions are out of phase, with the inertial actions in the downwind direction and the wave/current actions, represented by the wave trough actions, in the up-wind direction. $PHASE_{(-180)}$ is the reverse; the inertial actions are up-wind and the maximum wave/current actions are down-wind.

NOTE 5 The same total vectored sum of the actions and moments that comprise $(E_e + \gamma_{f,D}\, D_e)_{(a)}$ is used in Formulae (A.10.5-4) to (A.10.5-6). In effect, the base shear and overturning moment are the same for all of the first three cases: Formulae (A.10.5-4) to (A.10.5-6). This is because the load cases are designed to represent different interpretations of the same results from the random time domain dynamic analysis. When assessing the results of such an analysis, a procedure is used to capture both the maximum base shear and the maximum overturning moment. It is possible that the relationship between these two values is not known (i.e. the maximum base shear can occur at a different time than the maximum overturning moment). It is, however, known that the values of both items are maximized. MPMEs are then calculated by the method of choice, and $K_{DAF,RANDOM}$ values are calculated for base shear and overturning moment. These DAFs are well defined, but it is not necessarily known of what components they are comprised. The intent of Formulae (A.10.5-4) to (A.10.5-6) is to present three different sets of actions that can result in the different maxima base shear and overturning moments. Large correcting moments are likely to be necessary in Formula (A.10.5-5), the inertia-only load case. In Formula (A.10.5-5), the point of application of the actions has effectively moved from being predominantly close to the waterline (due to wave/current) with a relatively small inertial component at the hull centre of gravity to having the predominant action applied at the hull centre of gravity. Given that the hull centre of gravity is significantly higher than the point of application of the wave/current action and the requirement to have a consistent base shear and overturning moment, the introduction of large correcting couples at the hull is likely to be necessary.

NOTE 6 The base shear and overturning moments are the same in Formula (A.10.5-4) to (A.10.5-6), so there are unlikely to be significant differences in global jack-up response. The importance of the different load cases is the location of the actions and the components that comprise them. This can result in different member loads and stresses.

NOTE 7 Formula (A.10.5-7) can have a different combined base shear and overturning moment than Formulae (A.10.5-4) to (A.10.5-6). In Formula (A.10.5-7), the magnitude of $\gamma_{f,D}\, F_{in,PHASE(-180)}$ is identical, for both base shear and overturning moment, to the value of $\gamma_{f,D}\, F_{in,PHASE(180)}$ in Formula (A.10.5-6), but it is applied in the opposite direction. This case represents the wave/current actions in the down-wind direction and the inertial actions in the up-wind direction. In most cases, the magnitude of the vector $(E_e + \gamma_{f,D}\, D_e)_{(-180)}$ is smaller than the magnitude of the equivalent vector in Formulae (A.10.5-4) to (A.10.5-6). It is, however, possible that the internal leg stresses can be higher due to changes in internal leg shear and bending moments.

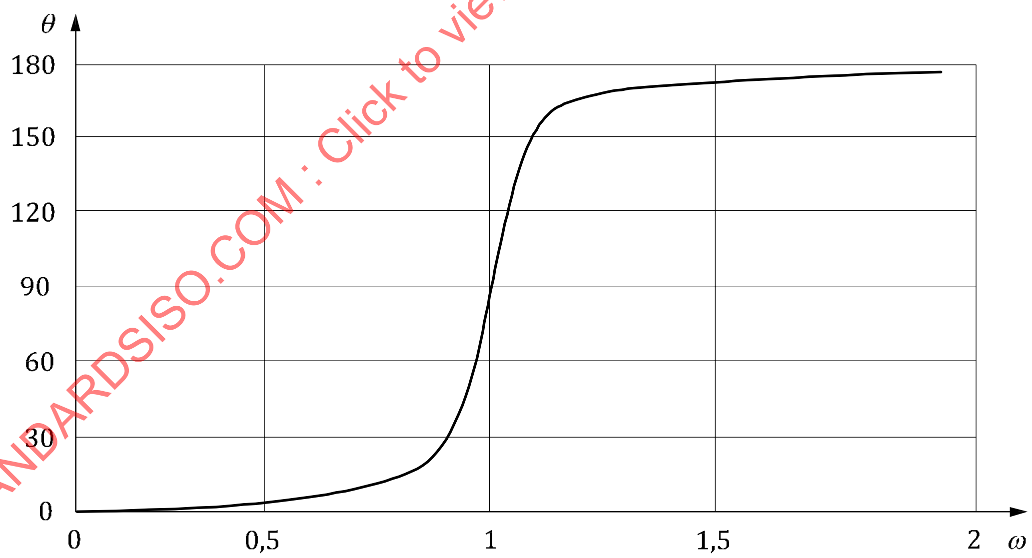


a) Representation of Formula (A.10.5-4) b) Representation of Formula (A.10.5-5) c) Representation of Formula (A.10.5-6) d) Representation of Formula (A.10.5-7)

Key

1 direction of storm
2 wind action F_{WIND}
3 wave action F_{STATIC}
4 up-wind wave action at wave trough $F_{STATIC.UP}$
5 inertial loadset $F_{in,PHASE(0)}$
6 inertial loadset $F_{in,PHASE(90)}$
7 inertial loadset $F_{in,PHASE(180)}$
8 inertial loadset $F_{in,PHASE(-180)}$ with magnitude of base shear and overturning moment equal to $F_{in,PHASE(180)}$ but applied in the opposite direction
9 simplified representation of wave/current action on jack-up
10 simplified representation of inertial action on jack-up
11 line indicating relative phase of wave/current action and inertial action for $(a) = (0)$
12 line indicating relative phase of wave/current action and inertial action for $(a) = (90)$
13 line indicating relative phase of wave/current action and inertial action for $(a) = (180)$
14 line indicating relative phase of wave/current action and inertial action for $(a) = (-180)$

Figure A.10.5-2 — Diagrammatic representation of the load cases given in Formulae (A.10.5-4) to (A.10.5-7) with the jack-up schematics showing the actions and the lower curves showing the phase between wave/current action and inertial action



Key

ω ratio of the natural period, T_n, to the period of the forcing action, T_f
θ phase angle in degrees between the forcing action and the inertial action

Figure A.10.5-3 — Phase between the forcing action and the inertial action for an SDOF system for varying ratios of natural period to forcing period (T_n/T_f) and damping of 7 % of critical

A.10.5.3 Stochastic storm analysis

A.10.5.3.1 General

In a stochastic storm analysis the extreme response can be predicted by stochastic methods where the intent is to determine the MPME of the responses of interest using statistical methods (see A.10.5.3.4). In the two-stage deterministic storm analysis, the MPMEs of the base shear and overturning moment are used to develop DAFs. For a one-stage stochastic storm analysis, the intent is to determine time histories of the utilizations from which the MPME utilizations can be calculated; see Figure 10.5-4.

In all stochastic analyses all action factors are set to 1,0 (see 8.8.1.3). When the stochastic storm analysis is used to determine a DAF (the first stage of a two-stage analysis), the metocean actions are unfactored in both the dynamic and the quasi-static analyses; the appropriate metocean action factor, $\gamma_{f,E}$, is applied in the second stage. However, when undertaking a fully integrated one-stage dynamic stochastic storm analysis that directly results in a time history of structural and foundation utilizations, the metocean parameters (i.e. wind velocity, wave height and current velocity) are factored; see A.10.5.3.2.

The waves can be modelled using a random superposition model, which is fully described in A.7.3.3.3.2, that identifies important constraints associated with this method of random wave dynamic analysis.

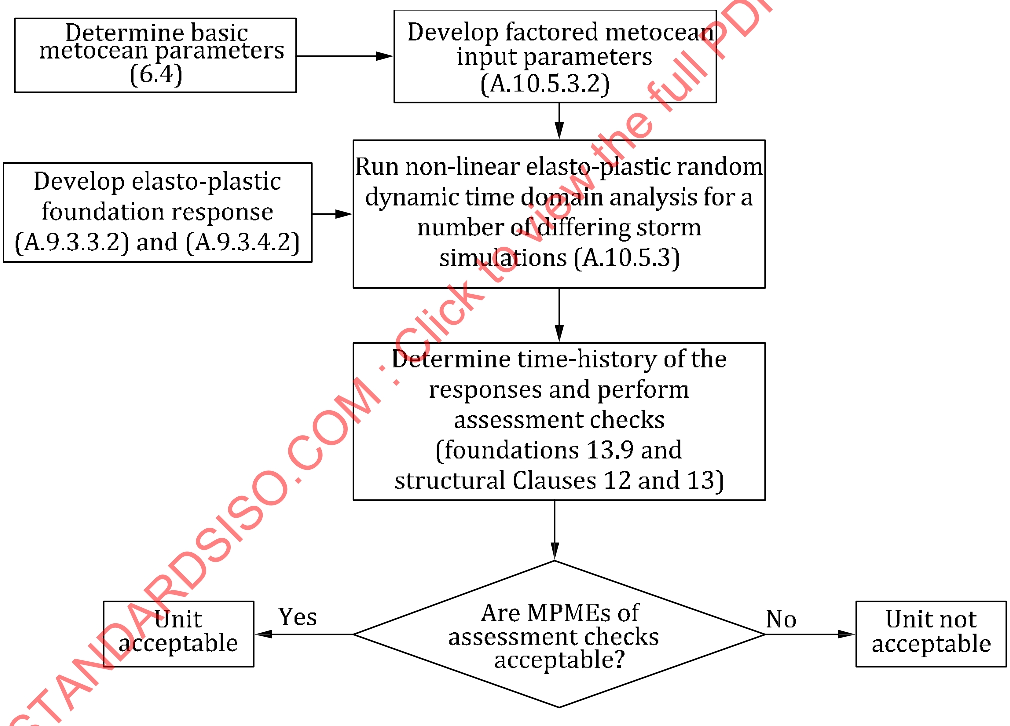


Figure A.10.5-4 — Analysis procedure for one-stage assessment with foundation fixity — Option 2

A.10.5.3.2 Application of partial factors to metocean parameters

When undertaking a one-stage fully integrated dynamic stochastic storm analysis, partial factors are applied to the metocean parameters. To ensure consistency between the one-stage stochastic and the two-stage deterministic approaches, the partial factors on metocean parameters should produce metocean action levels comparable to the factored quasi-static metocean actions used in the deterministic method.

When using dynamic stochastic storm analyses to determine a DAF for application in a two-stage deterministic analysis, the partial factors should be set to unity.

The partial factors on metocean parameters for fully integrated one-stage dynamic stochastic storm analyses can be determined as follows.

- Partial factor on wind velocity: The wind velocity used when generating the applied actions in accordance with A.7.3.4.1 should be factored by
  - $\sqrt{1{,}15}$ if 50 year return period independent metocean extreme storm actions are used, or
  - $\sqrt{1{,}25}$ if 100 year return period joint probability metocean data are used.
- Partial factors on wave height and current velocity: The partial factors for wave height and current velocity for use in the stochastic analysis are determined through an iterative process. The process involves factoring the wave height and current velocity until the metocean parameter-factored quasi-static stochastic wave/current action matches the action-factored quasi-static deterministic wave/current action computed using higher-order wave theory (see NOTE below). The effects of wave spreading (see A.6.4.2.8) should be consistently included or consistently excluded in the stochastic and deterministic calculations used in the calibration. As a first approximation, the same partial factors can be used as given above for wind velocity. Some adjustment can be necessary to achieve a good or conservative match between the following two pairs of action values:
  - the stochastic MPME and the deterministic maximum;
  - the stochastic mean and deterministic mean, the latter determined from integration over a full wave cycle (i.e. not from the average of the maximum and minimum values).

The match of MPME/maximum and mean actions is necessary to capture the cyclic behaviour. The adjustment generally results in different partial factors for the wave height and current velocity.

The wave period used in the stochastic analysis should be modified to maintain the same wave steepness as that of the unfactored sea state.

For the two-stage approach, the reference level for the wave and current actions is the quasi-static deterministic action. This reference level action is then modified through a DAF and the action factor to arrive at the final factored action. The important point is that the final action is founded on the quasi-static wave/current deterministic action. Conversely, in a fully integrated single stage analysis, there is no simple equivalent reference. It is, therefore, necessary to determine a stochastic equivalent to the factored deterministic quasi-static wave-current action. This is achieved by calculating the stochastic actions over three hours until partial metocean factors are found that match the MPME and mean actions with those from the action-factored quasi-static deterministic analysis. These partial metocean factors can, then, be used in the fully integrated stochastic dynamic analysis.

A.10.5.3.3 Random wave dynamic analysis method

Time domain simulations require that a suitable random sea state is generated, that the validity of the generated sea state is checked, and that the time step for the solution of the formulae of motion is sufficiently small. It is also necessary to ensure that the duration of the simulation(s) is sufficient for the method being used to determine the MPME. Specific recommendations are given in Tables A.7.3-3 and A.10.5-1.

Wave spreading may be taken into account, either by using a three-dimensional analysis method or by using the kinematics reduction factor in a two-dimensional analysis (see A.6.4.2.3). Accounting for wave spreading generally results in a smaller DAF.

A.10.5.3.4 Methods for determining the MPME

The extreme response that should be checked in the assessment is the MPME response which has a 63 % chance of exceedance in a three-hour storm. This MPME response is defined in Table A.10.5-1 as the mode value or highest point on the PDF. The stochastic waves modelled using a random superposition model result in non-Gaussian responses.

Four methods for obtaining the MPME of the response are included in Table A.10.5-1. Considerable care should be taken when T_n/T_p is greater than 0,8 and the use of any method to determine the MPME response should be critically assessed. When $T_n/T_p > 0,8$ other T_n/T_p ratios should be considered. The intent is to maximize the relevant responses (see A.10.4.2), but while not being unnecessarily conservative. This can be done by

— assessment of other wave height and period combinations (see A.6.4.2.9), or

— including or changing the level of spudcan fixity.

For the two-stage random dynamic analysis procedure the ratio of MPMEs of the dynamic to the quasi-static BS and OTM are used to determine the DAFs that are used to calculate the inertial loadset (see A.10.5.2.2.3).

Table A.10.5-1 — Recommendations for determining MPME

Method	Recommendations
General	The MPME is defined as the extreme with a 63 % chance of exceedance (typically this is the mode or highest point on the PDF). This is approximately equivalent to the 1/1000 highest peak level in a three-hour storm and the extreme with approximately a 63 % chance of exceedance.
Determination of the MPME from time domain simulations	Fit a Weibull distribution to the distribution of response maxima and determine the maximum value for the probability level of one exceedance in 3 hours. Take results as the average of MPMEs from at least 5 simulations. Each input wave simulation should be of sufficient length (usually more than 60 min, see Table A.7.3-3). See C.2.1. or Use multiple three-hour simulations and fit a Gumbel distribution to the absolute maximum from each simulation. Sufficient simulations (usually 10 or more) should be used to obtain stable MPME response values. See C.2.2. or Use Winterstein's Hermite polynomial model; when the kurtosis is > 5 use the improvements proposed by Jensen. Simulations of sufficient duration to provide stable skewness and kurtosis of responses (normally in excess of several hours). See C.2.3. or Use the drag-inertia method with appropriate scaling based on period ratio, to determine the DAFs for use in a two-stage deterministic storm analysis. Simulations of sufficient duration to obtain stable standard deviation of responses are required (usually more than 60 min). See C.2.4.

A.10.5.4 Initial leg Inclination

The effects of initial leg inclination should be considered. Leg inclination can occur due to leg-to-hull clearances and hull inclination. Generally, hull inclination limits are set in the operations manual. The total horizontal offset due to leg inclination, O_T, can be estimated as given in Formula (A.10.5-8):

$$O_T = O_1 + O_2 \quad \text{(A.10.5-8)}$$

where

O_T is the total horizontal offset of the leg base with respect to the hull;

O_1 is the offset due to leg-to-hull clearances;

O_2 is the offset due to maximum hull inclination permitted by the operating manual.

If detailed information is not available, O_T should be taken as 0,5 % of the leg length below the lower guide.

It is necessary to account for the effects of leg inclination only in structural strength checks. This can be accomplished by increasing the effective moment in the leg at the lower guide by an amount equal to the offset O_T times the factored vertical reaction at the leg base due to fixed, variable, environmental, inertial and P-Δ actions.

A.10.5.5 Limit state checks

The ULS responses for assessment should be determined using appropriate combinations of actions due to fixed and variable load, wave/current actions and wind actions as required by the acceptance criteria in Clause 13. The application of actions is described in 8.8; 5.4.3 requires that the analysis be carried out for a range of headings with respect to the jack-up such that the most onerous force(s) for each item listed in Table A.10.5-2 is(are) determined. The relevant ULS response parameters (action effects) are indicated in Table A.10.5-2.

Table A.10.5-2 — Action effects for limit state checks

Limit state check	Clause/Subclause	Response parameters(s)[a]	Action effect				
			G_F	G_V[b] min	G_V[b] max	E_e	D_e
Strength of members	12, 13.3	Member force vectors[c]	Y	Y[d]	Y	Y	Y
Spudcan strength	13.4	Forces on the spudcan	Y		Y	Y	Y
Holding system	13.5	Holding system force vectors	Y	Y[d]	Y	Y	Y
Overturning stability	13.8	Overturning moment				Y	Y
		Stabilizing moment	Y[e]	Y[d,e]			
Foundation capacity:	13.9, 9.3.6						
— preload	A.9.3.6.2	Vertical leg reaction	Y		Y	Y	Y
— sliding							
— pinned	A.9.3.6.3 or A.9.3.6.4.2	Vertical and horizontal leg reactions	Y	Y		Y	Y
— with moment fixity	A.9.3.6.5 (A.9.6.3.4.2)	Vertical, horizontal (and moment) leg reactions	Y	Y		Y	Y
— bearing							
— pinned	A.9.3.6.4.1	Vertical and horizontal leg reactions	Y		Y	Y	Y
— with moment fixity	A.9.3.6.5	Vertical, horizontal (and moment) leg reactions	Y		Y	Y	Y
— displacement	A.9.3.6.6	Spudcan displacements and reactions	Y	Y[f]	Y[f]	Y	Y

Key

Y = yes

G_F = actions due to the fixed load positioned such as to adequately represent the vertical and horizontal distribution

G_v = actions due to maximum or minimum variable load, as appropriate, positioned at the most onerous centre of gravity location applicable to the configurations under consideration

E_e = metocean action due to the extreme storm event

D_e = equivalent set of inertial actions representing dynamic extreme storm effects

[a] In all instances the responses are assessed including the effects of deformation under functional actions (hull sag) and large displacement (P-Δ) effects.

[b] Placed at most onerous centre of gravity position.

[c] The effects of leg inclination to be included, which may be added after global response analysis (see A.10.5.4).

[d] Consider minimum variable load if this is more onerous.

[e] Fixed and variable load are included in response calculation so that P-Δ effects are captured.

[f] As appropriate for the case under consideration: maximum for bearing and minimum for sliding.

A.10.6 Fatigue analysis

For jack-up operations of relatively long duration, see Clause 11.

A.10.7 Earthquake analysis

A.10.7.1 General

The provisions in A.10.7 complement ISO 19901-2 by presenting special aspects of an earthquake assessment procedure for jack-ups. The general procedures in Clauses 6 to 10 with the associated guidance in Annex A remain valid where appropriate; more specific reference to earthquake situations is provided in 6.6, 7.7, 8.6.3, 8.8, 9.4 and 10.3.

The greatest structural threat to a jack-up subjected to an earthquake is likely to be associated with vertical excitations that result in uneven settlement of the spudcans, which can cause lateral instability of the jack-up.

In situations where the jack-up is working over a platform, the relative motions between the platform and jack-up should be evaluated. The relative motions can affect the conductor and should be considered.

A.10.7.2 Earthquake assessment procedure

ISO 19901-2 gives alternative procedures for determining earthquake actions and alternative methods for the evaluation of earthquake activity. The selection of the procedure and the method of evaluation depend on the seismic risk category (SRC). The SRC depends on the exposure level and seismic zone in which the jack-up is to be located and is given in ISO 19901-2. The effects of near-source excitation should be considered (see A.10.7.5).

The simplified ELE screening methodology is given below and steps a) and b) are summarized in Table A.10.7-1.

a) Determine earthquake actions using either the simplified earthquake action procedure or the detailed earthquake action procedure specified in ISO 19901-2 to develop spectral response accelerations for a bedrock base. Use of the simplified procedure (maps) for the initial screening of jack-ups is encouraged.

b) Evaluate earthquake activity and the associated response acceleration spectra for the assessment of a jack-up against excitation of its base by ground motions using either ISO maps, regional maps or a site-specific earthquake hazard analysis, as specified in ISO 19901-2. Since ISO map accelerations are for a 1 000 year return period on rock, adjust the spectral shape for the 1 000 year event as described in ISO 19901-2 at the spudcan depth as a function of site soil characteristics.

c) Perform response spectrum analysis in accordance with A.10.7.3.

d) Evaluate the performance of the jack-up using the ULS assessment procedures provided in Clause 13.

e) If the jack-up does not pass the simplified procedure, proceed to a more detailed assessment in accordance with A.10.7.4 using alternative analysis methods (see 10.9) and the ISO 19901-2 ALE procedures.

Table A.10.7-1 — Simplified procedure to develop 1 000 year ELE screening spectra using ISO 19901-2

Item	Source in ISO 19901-2:2022
1 000 year accelerations – determine $S_{a,map}$	Annex B or site-specific
Determine the site seismic zone	6.4a
Simplified seismic action procedure	Clause 7
Determine soil site class	7.1a and Table 5
Spectral parameters C_a and C_v	7.1b and Tables 6 and 7
Develop horizontal spectrum	7.1c and Figure 2
Develop vertical spectrum	7.1d and Figure 3
Select damping	7.1e – use 5 % unless an alternate value justified
Seismic action procedure	7.2, $N_{ALE} = 1$ and $C_{r=1}$

A.10.7.3 ELE assessment

A.10.7.3.1 Partial action factors

The foundation, leg members and leg-to-hull connection should be assessed for the factored assessment actions defined in 8.8.1 for earthquake situations. The inertial action induced by the ELE ground motions should be determined using dynamic analysis procedures such as response spectrum analysis or time history analysis.

NOTE Reference can be made to Annex B, which contains all of the applicable partial action and resistance factors for a site-specific analysis.

Spudcan sliding should be considered for the minimum vertical reaction (uplift case) when the earthquake actions oppose the weight.

A.10.7.3.2 Structural and foundation modelling

The mass used in the dynamic analysis should consist of the mass of the structure associated with

- the fixed load G_F,
- the best estimate of the variable actions; in lieu of specific data, 75 % of the maximum variable load G_v can be used,
- the mass of entrapped water in the leg components and spudcans, and
- the added mass.

The added mass can be estimated as the mass of the displaced water for motion transverse to the longitudinal axis of individual structural members and appurtenances (see A.7.3.2). Table A.7.3-2 gives the hydrodynamic inertia coefficients that should be used in an earthquake assessment. For motions along the longitudinal axis of the structural members and appurtenances, the added mass may be neglected (except for spudcans). The spudcan internal entrapped mass should be included in the mass model and the spudcan added mass (surrounding water and/or soil) should be included where significant.

The structural model should include the three-dimensional distribution of the stiffness and mass of the structure and account for large-displacement effects.

The vertical distribution of mass is important for earthquake analyses as it affects the lateral inertial actions. Care should be taken when modelling the hull mass to ensure that the horizontal distribution of mass is correct as it affects the yaw response. The cantilever position should be considered when distributing the mass.

Asymmetry in the distribution of the stiffness and mass of the jack-up can lead to significant torsion and should be considered in the assessment. The jack-up model should represent the operational configuration but the effects of the drill string can be ignored. Where the jack-up is supporting more than one conductor, their mass, added mass and stiffness should be considered in the model.

For earthquake assessments, the hull sag moment should be based on the operating condition (see A.8.8.3).

In computing the dynamic characteristics of the jack-up, a modal damping ratio of up to 5 % of critical may be used in constructing spectra for the ELE event. In addition, for the primary vertical mode, radiation damping in accordance with A.10.4.3.4 can be included in the vertical response spectra definition. Additional damping, including hydrodynamic or soil induced damping (hysteretic and radiation), should be substantiated by special studies. Guidance on structural damping, hydrodynamic damping, foundation damping and vertical radiation damping are given in A.10.4.3.

Soil springs derived from small strain initial stiffnesses should be used to determine the natural periods.

For earthquake screening analyses, the simplest adequate spudcan-soil models should normally be used. These models should incorporate the maximum interpreted small strain stiffnesses and maximum capacities (see Clause 9 and A.10.4.4.2). Soil stiffness degradation should not normally be included in an earthquake screening analysis. More detailed spudcan-soil interaction representations may be used.

The minimum soils information should be obtained in accordance with A.6.5, but to a depth of 2 diameters below the deepest spudcan penetration. Depth to bedrock or a competent soil layer is required and can be estimated from geophysical investigation data or regional considerations.

Foundation performance should be determined on the basis of studies that consider the assessment actions. Except for the simplified screening analysis, the non-linear stiffness and capacity of the foundation should be addressed in a manner compatible with Clause 9. If uplift or sliding is indicated from the screening analysis, non-linear dynamic time history or pushover analyses can be used to evaluate cumulative displacements and the resulting structural condition.

Vertical actions on the foundations should not normally exceed the preload. If the vertical actions on the foundations exceed the preload and the ULS Step 3 displacement check (see A.9.3.6.6) reveals the potential for excessive additional penetration, non-linear dynamic time history analyses with cyclic degradation can be used to evaluate cumulative displacements and the resulting structural condition, e.g. encroachment on an adjacent fixed platform.

A.10.7.4 ALE assessment

For jack-ups that do not satisfy the ULS criteria for the ELE screening assessment, a site-specific non-linear ALE assessment can be used to try and demonstrate acceptability. This can be satisfied by a pushover analysis or by time history analyses using ALE excitation.

Where substantial spudcan settlement or liquefaction is a possibility, a fully non-linear cyclic degrading analysis using best available soils modelling technology is recommended.

A.10.7.5 Near-source excitation

If operating close to an active fault (typically within about 15 km), it can be necessary to consider near-source ground motions. At these near-source distances, the ground motions can exhibit substantial rupture directivity effects and directionality, with motion characteristics often considerably in excess of normal design values, including permanent offsets, larger amplitude ground motions at relatively longer periods (e.g. $T \geq 1$ s), and vertical motions equal to or greater than horizontal motions at shorter periods (e.g. $T \leq 0,3$ s).

A.10.8 Ice

A.10.8.1 General

A.10.8.1.1 Operating area types

For the purposes of this document, arctic and cold regions can be split into three types of area. These area types are not absolutely distinct, but have been delineated to give guidance on how the site-specific assessment should be undertaken. The three area types are:

Area type 1 Areas that have a well-defined ice-free season during which ice development is extremely low probability, and there is no possibility of large scale ice features. An example of Area type 1 is upper Cook Inlet after all the ice has melted in the Inlet, and the intrusion of ice from Gulf of Alaska is not possible.

Area type 2 Areas that have a well-defined season during which the area is free of sea ice, but can be affected by large scale ice features such as icebergs. An example of Area type 2 is the East Coast of Canada during the summer months.

Area type 3 Areas that can have ice free periods, but can be subject to ice, in certain years, at any time of the year (e.g. parts of the Chukchi Sea) and areas that are not normally free from ice for any extended period at any time of the year.

When deciding the classification of the proposed area of operations guidance should be sought from the operator and the relevant coastal state.

The unit should be operated in conformity with its Marine Operations Manual.

A.10.8.2 ULS

No guidance offered.

A.10.8.3 ALS

No guidance offered.

A.10.8.4 Assessments in the area types

A.10.8.4.1 Area type 1

Jack-ups operating in type 1 areas, during the ice free season should be assessed in accordance with this document.

Studies should be undertaken to define when the ice free season starts and ends (see ISO 19906 and ISO 35106). Operations should not commence until the area has been confirmed to be ice free. Ice detection and threat assessment should follow the guidelines of ISO 35104. The jack-up should be moved to a safe location before the start of the next ice season, even if planned operations have not been completed (see A.10.8.7). Plans, including contingencies, should be developed using risk analysis.

For an area type 1, the potential for ice encroachment from other areas (e.g. open ocean into a bay or inlet) should be assessed to have an annual probability of ice interaction less than 10^{-4}. Given this low probability of ice interaction [below the abnormal-level ice event (ALIE) requirements of ISO 19906] there is no requirement for establishment of physical ice management procedures. Since operations occur only when the area is free of ice and ALIE criteria are met, there is no extreme level ice event (ELIE) to be considered.

A.10.8.4.2 Area type 2

Jack-ups operating in type 2 areas should be assessed to this document, supplemented by an ice management system. The ice management system should be developed to ensure that the annual probability of interaction between ice and the jack-up is below the 10^{-4} (the ALIE annual probability given in ISO 19906:2019, 7.2.2.4).

Studies should be carried out to demonstrate that the ice management system can reduce the probability of iceberg interaction to below 10^{-4}. Iceberg detection procedures should be established to ensure there is sufficient time for iceberg management, see ISO 35104.

It should be demonstrated that the probability of sea ice is below the ALIE without recourse to physical ice management.

Ice management plans can include moving the jack-up off site. If moving the jack-up is part of the methodology for reducing the ALIE probability, then due account should be taken of potential delays (see A.10.8.6).

A.10.8.4.3 Area type 3

Jack-ups operating in type 3 areas should be assessed using the provisions relating to ice and actions contained in ISO 19906, ice management in ISO 35104, and supplemented by this document.

The applicable combinations of wind, wave, current and ice actions are set out in ISO 19906. The sources of the annual probabilities associated with the actions and action factors are given in 10.8.2 and 10.8.3. The resistance factors should be in conformity with this document. The type, dimensions and properties of the ice used for the ice actions should be in accordance with ISO 19906, accounting for the season of operation and ice management procedures per ISO 19906.

A detailed risk assessment should be conducted to determine, document and mitigate the ice related risks to the safety of the unit and its operation. Factors to be considered in such a risk assessment include, but are not limited to:

- the capability of the unit to locally and globally withstand the ice actions given in ISO 19906 (e.g. increased wave actions on braces with increased ice build-up and the effects of the accumulation of ice rubble within the leg);
- the reliability and effectiveness of ice forecasting and ice monitoring services, see ISO 35104 and ISO 35106;
- the reliability and effectiveness of ice management services, see ISO 35104;
- the ability to remove the unit from approaching iceberg tracks in a timely manner;
- the feasibility of transit to and from the area of operations within the allowable temperature and ice conditions limitations;
- the effects of ice accretion on the elevated and floating stability of the unit, see ISO 19906 and ISO 35106;

— reliability of emergency evacuation and rescue systems, see ISO 19906:2019, Clause 18 and ISO 35102.

Iceberg and sea ice detection procedures should be established that will ensure ice is detected far enough in advance for ice management to be effective, see ISO 35104.

Ice management plans can include the effects of moving the jack-up off site, but if moving the jack-up is part of the methodology for reducing the ALIE actions, then due account should be taken of potential delays (see A.10.8.5.5).

A.10.8.5 Additional factors to be considered for arctic and cold regions

A.10.8.5.1 General

As specified in A.10.8.1, jack-ups operating in arctic and cold regions should be assessed to this document and ISO 19906, as appropriate. Notwithstanding the general requirement for conformity with relevant clauses in these documents, some areas of specific consideration are discussed in A.10.8.5.2 to A.10.8.5.4.

A.10.8.5.2 Geotechnical and geophysical considerations

Geotechnical assessment should be carried out using information, penetrations, resistance factors, and additional penetration assessment calculated in accordance with Clause 9 of this document. However, due consideration should be given to arctic and cold region specific geophysical and geotechnical conditions with particular consideration given to the potential for permafrost at the site (see ISO 19906:2019, Clause 6; ISO 35106). The possible interaction between the spudcan and permafrost during and after preloading should be carefully considered taking full account of the potential for punch-through during preloading and sudden additional penetration during operations. The methodology for assessing permafrost given in ISO 19906 is not directly applicable to spudcan foundations, because they are small, but can be used to help establish a more spudcan-specific methodology.

The potential for strudel scour near the mouth of a river should be assessed (see ISO 19906, ISO 35106).

A.10.8.5.3 Dynamic ice actions

Dynamic sea ice actions (see ISO 19906) can affect jack-ups. The magnitude of the effect is currently unknown for jack-ups specifically and should be investigated, since structures that have experienced ice-induced vibrations have had natural frequencies in excess of most jack-ups (0,4 to 10 Hz versus a likely jack-up frequency of 0,1 to 0,3 Hz). The same ice failure mechanisms are likely to apply to jack-ups as for jacket structures and other slender/flexible structures, see Yue and Bi (2000)[213], Blenkarn (1970)[25], Määttänen (1975)[131] and Johnston et al. (1999)[113].

Wave induced velocity of ice features (see ISO 19906, 8.3.2) can adversely affect the structure of a jack-up and should be assessed.

A.10.8.5.4 Factors to consider for moving a jack-up off site in arctic and cold regions

Normal field moves of jack-ups are not part of the scope of this document, but in arctic and cold regions, when moving a jack-up off site is part of the specified contingency plans for jack-up elevated operations, some special considerations should be taken. The limits on jacking the unit down and moving off site can be more restrictive than the elevated limits of the jack-up. Under these circumstances, plans should be established to move the jack-up based on the jacking and move limitations, not the elevated limits.

Due consideration should be given to all the factors that could affect the ability of the jack-up to move off site. Some of the factors can include, but are not limited to:

- time to pull legs;
- effect of permafrost on leg extraction (e.g. possible adhesion of the spudcan to permafrost);
- potential for water freezing in the jetting lines, thereby making them ineffective;
- consequences of jacking iced leg/chords/braces through the guides in event of leg icing;
- potential for ice accumulation on top of the spudcan as leg is jacked up;
- jack brakes freezing;
- ability to skid the cantilever in;
- jacking the hull down onto ice features;
- ice actions on the hull while pulling legs;
- effects of ice accretion on the floating stability of the unit;
- towing speed when leaving site can be slower than in open water conditions;
- reliability of the tugs and other necessary equipment in arctic and cold regions;
- availability of a safe location to which to tow the jack-up.

The above list is not intended to be complete. It is intended to give some guidance on factors that should be considered. A risk assessment of the operation of moving the jack-up off site should be carried out prior to commencing site-specific operations on the site. The risk analysis should consider all the factors that could adversely affect the ability and timing of a move. Any high risks identified during the risk assessment should be suitably mitigated.

A.10.9 Accidental situations

No guidance is offered.

A.10.10 Alternative analysis methods

A.10.10.1 Ultimate strength analysis

No guidance is offered.

A.10.10.2 Methodology

When using the provisions of ISO 19902:2020, 7.11 (reserve strength), care should be taken in modelling non-linear behaviour of chords and holding system of a jack-up structure.

A.11 Guidance on long-term applications

A.11.1 Applicability

No guidance is offered.

A.11.2 Assessment data

A.11.2.1 Jack-up data

A list of relevant modifications should be compiled including information about weights, wind areas and appurtenances added or removed that affect mass, applied actions and structural integrity.

For a long-term application, such modifications can typically include:

— increased weight and wind area from such items as production modules, risers, flare towers, accommodation blocks, and conductors;

— increased wave and current actions due to risers, conductors or other structures exposed to waves.

A.11.2.2 Metocean data

The data required for a fatigue analysis should include long-term wave data in the form of a wave scatter diagram or a table of representative sea states, refer to A.6.4.2.10.

Joint probability and/or directional metocean data can be used to optimize the ULS and FLS assessment for the long-term application.

A.11.2.3 Geotechnical data

Effects of seabed scour, differential settlement, consolidation settlement, expected reservoir subsidence, sand waves, etc. can be of greater significance for long-term applications. For this reason, the site-specific geotechnical data should include the information necessary to evaluate these phenomena.

A.11.2.4 Other data

Further data associated with the long-term application can be required. Examples include the possible effect on geotechnical properties due to top-hole construction activities, marine growth, effects from adjacent structures, etc.

A.11.3 Special requirements

A.11.3.1 Fatigue assessment

A.11.3.1.1 Historical damage

The assessment should take into account the fatigue history of critical details prior to installation on the planned site and focus on details of member connections that are essential to the overall structural integrity of the jack-up. In order to assess existing fatigue damage, specific information relevant to prior installations is required. The availability of the information depends on the information collected and retained by the jack-up owner over the life of the jack-up. The quality of the database affects the historical results. The historical data can have a large variability, requiring the assessor to make assumptions in the historical fatigue assessment. The assessment can include detailed fatigue analysis of the historical data and/or evaluation of inspection records. Parameters identified as important in addressing the historical aspects of jack-up fatigue are as follows:

— geographic region (e.g. Gulf of Mexico, North Sea, Eastern Canada) and, where available, the co-ordinates of the previous sites so that metocean parameters can be developed for use in historical analysis;

— hull elevation and orientation;

— water depth;

— penetration;

— soil type and characteristics.

A.11.3.1.2 Fatigue sensitive areas

Areas that are susceptible to fatigue damage include

— leg members and joints in the vicinity of the upper and lower guides for the operating leg/guide location,

— leg-to-hull holding system,

— leg members and joints adjacent to the waterline,

— leg members and joints in the lower part of the leg near the spudcan, and

— spudcan-to-leg connection.

Normally, it is not necessary that the fatigue assessment include consideration of the hull structure since the long-term cyclic loading is similar to that experienced in multiple short-term operations. Generally the hull is not fatigue sensitive.

A.11.3.1.3 Special considerations for fatigue assessment

Special considerations in the fatigue assessment are listed below.

— Inclusion of detailed models to arrive at local stress levels:

Areas in the structure with high stress levels can be identified using models developed for global analysis and the stress ranges determined using appropriate stress concentration factors (SCFs) from literature. Alternatively, more detailed fine-mesh finite element models can be used to determine the hotspot stress ranges [suitable methodologies are given in DNV-OS-C104 (DNV 2022a)[63], DNV-RP-C203 (DNV 2021c)[59], ABS 115 (2003a)[11], ABS 115 (2003b)[12], API RP 2A-LRFD (1993)[14], HSE (2001)[91] and HSE (1999)[90]].

— Effect of foundation stiffness (seabed fixity):

The stiffnesses of the foundations are a function of the soil properties, the strain amplitudes and loading history (see A.9.3.4). As a consequence, the foundation modelling should consider upper and lower bound stiffnesses (see A.9.3.4.3 for clay and A.9.3.4.4 for sand). Typically, the fatigue assessment of the spudcan and lower part of the leg requires the use of upper bound stiffness, while the fatigue assessment for the upper leg and the leg-to-hull interface requires lower bound stiffness. Although the foundation stiffness varies as a function of the reactions beneath the spudcan, the variation is unlikely to be of significance except, possibly, for low-cycle fatigue.

— Inclusion of non-linearities and dynamics:

The structural response of a jack-up is such that pure linear techniques can be inadequate. Therefore, the analysis should include the non-linear effects of the structure. These can include

— hydrodynamic actions,

— large displacement effects (see 8.8.6),

- dynamic amplification (see 10.5.2, 10.5.3), and
- leg-to-hull interface, e.g. ensuring that those structures that transfer force in compression contact only are properly modelled.

A.11.3.1.4 Fatigue analysis methodology

A robust analysis method should be used to determine the fatigue damage. The method should determine the response of the jack-up structure to various sea states representing the operational environment. The jack-up should be considered in the operational configuration, which includes the levels of variable load, hull elevation and cantilever position.

Wave spreading and directionality effects can be included.

Foundation stiffnesses are generally assumed to be linear in smaller sea states. A check of non-linearity should be performed to validate this assumption for higher sea states.

For guidance on suitable fatigue analysis methodology, S-N curves and SCFs the assessor is referred to one of the integral methods outlined in Table A.11.3-1. These should be used taking account of the specific structural characteristics of the jack-up as described above.

For fatigue analysis the partial action factor should be reduced to unity when using S-N curves at the mean minus two standard deviations of log(N).

Table A.11.3-1 — Sources of guidance on fatigue analysis methodology

Organization	Document	Reference
DNV	Methods are given in DNV-OS-C104 Technical guidance on fatigue calculations, e.g. calculation methods, SN-curves, SCFs are given in DNV-RP-C203 Fatigue design of offshore steel structures	DNV-OS-C104 (2022a)[63] DNV-RP-C203 (2021c)[59]
ABS	Methods are given in the Guide for the Fatigue Assessment of Offshore Structures (April 2003) Commentary on the Guide for the Fatigue Assessment of Offshore Structures (April 2003)	ABS 115 (2003a)[11] ABS 115 (2003b)[12]
API	Methods are given in API RP2A-LRFD-2019	API RP 2A-LRFD (2019)[14]
UK HSE	Guidance is given in OTO 2001/015 and OTH92 390	HSE (2001)[91] HSE (1999)[90]
ISO	Methods are given in ISO 19902	—

A.11.3.1.5 Fatigue acceptance criteria

The fatigue analysis should determine the fatigue damage in the period before, as well as during the long-term application of the jack-up. The margin of safety of a structural detail depends on its accessibility for inspection and the availability of one or more alternative load paths (redundancy) after failure of the detail investigated. The acceptance criterion for fatigue strength is as given in Formula (A.11.3-1):

$$f_{\text{FD,e}} D_{\text{c,e}} + f_{\text{FD,s}} D_{\text{c,s}} < 1{,}0 \quad \text{(A.11.3-1)}$$

where

$D_{c,e}$ is the calculated existing fatigue damage prior to arriving at the site;

$D_{c,s}$ is the calculated fatigue damage during planned operations on site;

$f_{FD,e}$ is the fatigue damage design factor applicable to $D_{c,e}$; generally, $f_{FD,e} = f_{FD,s}$, but $f_{FD,e}$ should not be taken larger than 2 if the detail has been inspected thoroughly before the long-term application;

$f_{FD,s}$ is the fatigue damage design factor applicable to $D_{c,s}$; see Table A.11.3-2 or A.11.3-3.

Table A.11.3-2 — Fatigue damage design factor $f_{FD,s}$

Fatigue damage design factor, $f_{FD,s}$	Full access for inspection and repair	Access for inspection, no repair during operation	No access for inspection, no repair during operation
Full redundancy/minor consequence	2	3	5
No redundancy/major consequence	3	5	10

The values in Table A.11.3-3 give more detailed guidance for structures that are fully redundant, i.e. the structure does not have single members or member connections that, when damaged, can cause a failure with major consequence. This is typical of RCS approved jack-ups with braced legs.

Table A.11.3-3 — Fatigue damage design factor $f_{FD,s}$ — Redundant structure

Description (assumes there is structural redundancy for every member and member connection)			Fatigue damage design factor, $f_{FD,s}$
Can inspect and repair	Hull structure	Primary hull structure	1
		Leg-to-hull interface structure with access for inspection and repair	2
	Leg structure in air	Leg chords, brace to chord joints, brace joints	2
Can inspect but not repair	Leg structure in splash zone	Leg chords, brace to chord joints, brace joints	3
	Leg structure under water	Leg chords, brace to chord joints, brace joints, leg to spudcan connection	3
	Spudcan	Structure with access for inspection and repair	3
Cannot inspect or repair	Hull structure	Leg-to-hull interface structure without access for inspection and repair	5
	Leg structure under sea floor	Leg chords, brace to chord joints, brace joints, leg to spudcan connection	5
	Spudcan	Structure without access for inspection and repair	5

If necessary, fatigue life enhancement methods such as weld profiling, weld toe grinding and peening may be used, subject to RCS approval. Peening should only be used for improving fatigue lives after appropriate inspection.

A.11.3.2 Weight control

A weight control procedure should be prepared by the party responsible for operating and maintaining the jack-up during the long-term application. The procedure should be used to track the changes in weights and to ensure ongoing conformity with the assumptions used in the assessment.

The weight control procedure should be sufficient to satisfy the RCS requirements in lieu of the periodic dead weight survey. This should include wet weights where applicable.

A.11.3.3 Corrosion protection

No guidance is offered.

A.11.3.4 Marine growth

Marine growth should be taken into account in the site-specific assessment. The assessment can be for either the growth specified for the application period or for a pre-determined limit. In either case, the actual growth should be monitored and, when necessary, removed to ensure conformity with the assessment assumptions.

A.11.3.5 Foundations

Settlements can occur (see A.9.3.6.7, 11.3.5 and A.11.2.3), resulting in the loss of air gap or the hull being out-of-level. The consequences of resolving these should be considered in the assessment, e.g. the effect of guide position on the fatigue or strength analyses, changes in conductor support, etc.

Consolidation of the soil through dissipation of pore pressures during the long-term operation can result in changes in foundation strength and stiffness. This affects the redistribution of leg moments and changes the dynamic response. The effects on fatigue life and strength should be considered, especially at the leg to spudcan connection.

In conditions where scour can occur, scour protection can be required.

A.11.4 Survey requirements

A.11.4.1 Pre-deployment inspection plan

The RCS special survey requirements prior to a long-term application can be more extensive than those of a typical special survey. Therefore, it is advisable to plan the surveys prior to mobilisation to a shipyard for modifications. The inspection plan should specify the locations and types of inspection, taking into account the areas that the assessor has identified as being critically stressed during the extreme storm or being fatigue sensitive during the long-term application. Areas that are not accessible, or are difficult to access for in-service inspection, should be subject to more detailed pre-deployment inspection and should be specially evaluated (see A.11.3.1).

A.11.4.2 Project specific in-service inspection programme

The project specific in-service inspection programme (PSIIP) should be developed by modifying and updating the existing in-service inspection programme normally required by the RCS. The PSIIP should reflect the requirements for the planned long-term application.

NOTE The PSIIP is likely to be subject to direction and approval by the RCS.

Areas that require special inspection procedures, such as underwater parts, should have documented inspection procedures, giving due consideration to the most suitable and practical methods.

The results of the in-service inspections should be reviewed and, if appropriate, the PSIIP modified to reflect the results of this review. This information can be relevant to ensure the ongoing validity of the PSIIP and for extending the jack-up's time on site beyond that originally planned.

A.11.4.3 Alternative project specific in-service inspection programme (PSIIP)

An alternative can be derived using a probabilistic approach. The safety philosophy behind the alternative PSIIP should be in accordance with the RCS's safety philosophy and the structural reliability level inherent in the RCS rules should be maintained. The approach developed should be documented.

When using a probabilistic approach, it should be recognized that uncertainties are associated with prediction of the fatigue performance and the inspection techniques applied. Key uncertainties should be accounted for in the probabilistic analysis.

A.12 Guidance on structural strength

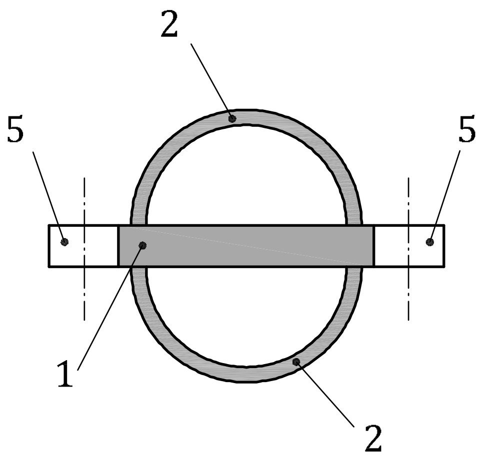
A.12.1 Applicability

A.12.1.1 General

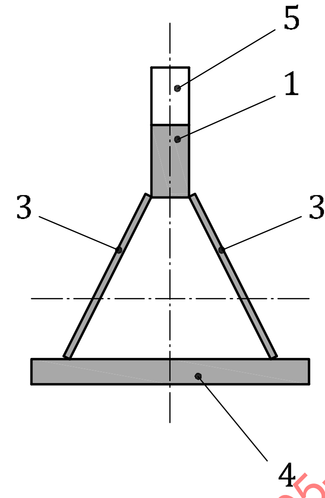
Clause A.12 applies to steel structures only. Where necessary, the formulae included in A.12 have been non-dimensionalized using Young's modulus, E, of 205 000 N/mm^2 (or 29 700 ksi).

For the purposes of strength assessment, it is necessary to consider the truss type leg structure as being comprised of structural members. Typically, each structural member can be represented by a single beam-column element in an appropriate analytical model of the structure. Examples of structural members are braces and chords in truss type legs and box or tubular legs, all of which form a part of the structure for which the properties can readily be calculated.

The cross-section of a non-circular prismatic structural member is usually comprised of several structural components. Table A.12.2-1 shows classification limits for circular and non-circular prismatic members in typical jack-up chords comprising split-tubulars, rack plates, side plates and back plates (see Figure A.12.1-1). A component is by definition comprised of only one material. Therefore, where a plate component is reinforced by another piece of plating of a different yield strength (see Figure A.12.2-1) the reinforcing plate should be treated as a separate component. Non-circular prismatic members should be assessed using the provisions of A.12.6.



a) Opposed rack split tubular chord member section



b) Triangular type chord member section

Key

1 rack plate
2 split tubular
3 side plate
4 back plate
5 rack tooth

Figure A.12.1-1 —Typical components of typical jack-up chord cross-sections

Tubulars should be assessed as structural members using the provisions of A.12.5.

In Clause 12, subscripts y and z are used to define the two axes of bending of tubular and prismatic members, however F_y is used to define the yield strength in stress units.

NOTE The structural resistance factors for tubular members given in Clause 12 are based on an independent interpretation of the theoretical values derived from the data used in the calibration of API RP 2A LRFD, 1st edition, to API RP 2A, 15th edition, and the data used in the development of the ISO 19902 tubular members strength formulations. The values for non-tubular prismatic members were taken from AISC; see American Institute for Steel Construction (AISC) (2005)[13], which changed its equivalent resistance factor from 1,18 to 1,1 between the 1986 and 2005 editions because of a reassessment of the applicable data, which resulted in an effective reduction in the coefficient of variation.

A.12.1.2 Truss type legs

No guidance is offered.

A.12.1.3 Other leg types

No guidance is offered.

A.12.1.4 Fixation system and/or elevating system

No guidance is offered.

A.12.1.5 Spudcan strength including connection to the leg

No guidance is offered.

A.12.1.6 Overview of the assessment procedure

No guidance is offered.

A.12.2 Classification of member cross-sections

A.12.2.1 Member type

No guidance is offered.

A.12.2.2 Material yield strength

The value of the yield strength taken from a tensile test should correspond to the 0,2 % offset value. Where this value is greater than 90 % of the ultimate tensile strength (UTS), the yield strength, F_y, used in A.12 should be taken as 90 % of UTS. The following variables are used in A.12:

F_y is the yield strength in stress units (minimum of the yield strength and 90 % of the UTS);

F_{yi} is the yield strength of the *i*th component of the cross-section of a prismatic member, in stress units (minimum of the yield strength and 90 % of the UTS of the *i*th component of the cross-section);

F_{ymin} is the minimum yield strength of the F_{yi} of all components in the cross-section of a prismatic member, in stress units;

F_{yeff} is the effective yield strength of the cross-section of a prismatic member, in stress units, determined from the plastic tensile axial strength divided by the minimum cross-sectional area.

A.12.2.3 Classification definitions

A.12.2.3.1 Tubular member classification

A cross-section of a tubular member is a class 1 section when Formula (A.12.2-1) applies:

$$D/t \leq 0{,}051\,7\;E/F_y \qquad \text{(A.12.2-1)}$$

where

D is the outside diameter;

t is the wall thickness;

F_y is the yield strength in stress units;

E is Young's modulus of steel ($E = 205\,000\ \text{N/mm}^2$).

NOTE Conformity with class 1 classification is relevant only when undertaking earthquake, accidental or alternative strength analyses (see 10.7, 10.8 and 10.9). In all other cases, the distinction between class 1 (plastic) and class 2 (compact) is irrelevant to the assessment.

A.12.2.3.2 Non-circular prismatic member classification

Non-circular prismatic members that contain curved or tubular components should have the curved components classified based on the values given in Table A.12.2-1 and their flat components classified based on Tables A.12.2-2 to A.12.2-4. The limits given in Table A.12.2-1 tend to be conservative as, in most cases, there is additional support for the curved component by the flat components (e.g. the rack in a split tube chord reinforces the split tube and helps to prevent local buckling). When the limits given in Table A.12.2-1 are considered to be too onerous, it can be possible to justify the use of alternative limits through rational analysis.

NOTE The use of Tables A.12.2-3 and A.12.2-4 to classify cross-sections subject to axial compression and bending is complicated and requires knowledge of the cross-section stress distribution. It is always acceptable to conservatively base the cross-section classification on the relevant axial compressive case.

Table A.12.2-1 — Classification limits for non-circular prismatic members containing curved components

Class	D/t limits	
	Section in bending	**Section in compression**
1	$D/t \leq 0{,}052\ E/F_y$	$D/t \leq 0{,}052\ E/F_y$
2	$D/t \leq 0{,}103\ E/F_y$	$D/t \leq 0{,}077\ E/F_y$
3	$D/t \leq 0{,}220\ E/F_y$	$D/t \leq 0{,}102\ E/F_y$
4	$D/t > 0{,}220\ E/F_y$	$D/t > 0{,}102\ E/F_y$

When classifying non-circular prismatic components in accordance with Table A.12.2-2 to A.12.2-4, a distinction is made between internal components and outstand components as follows:

a) internal components are components that are supported by other components along both longitudinal edges, i.e. the edges parallel to the direction of compression stress, and include

— flange internal components: internal components parallel to the axis of bending;

— web internal components: internal components perpendicular to the axis of bending;

b) outstand components are components that are supported by other components along one longitudinal edge and at both ends of the member under consideration, with the other longitudinal edge free.

When a cross-section is composed of components of different classes, it is classified according to the highest (least favourable) class of its compression components. Components within a cross-section can be ignored, provided that only the remaining cross-section is used for all aspects of the assessment. However, if a component that has been ignored is required to carry local loading, e.g. horizontal pinion thrust, the effects of the global actions should be considered when that component is assessed for the local loading. The effects of the global actions can normally be included by considering the global deformations of the member in addition to the local loading.

In calculating the ratios given in Tables A.12.2-2 to A.12.2-4, the dimensions that should be used are those given in the relevant table. The components are generally of constant thickness; for components that taper in thickness, the average thickness over the width of the component should be adopted.

Members that do not satisfy the applicable simplified lateral torsional buckling (LTB) criteria should be assessed further to determine a reduced representative member bending moment strength, M_b, using the guidance in A.12.6.2.6.

The LTB criterion for singly symmetric open sections is taken from F2-5 of AISC (2005)[13], as given in Formula (A.12.2-2):

$$\frac{L_b}{r_{ltb}} \leq 1,76\sqrt{\frac{E}{F_{y,ltb}}} \tag{A.12.2-2}$$

The LTB criterion for any closed section is derived from BS 5400-3, as given in Formula (A.12.2-3):

$$\frac{L_b}{r_{ltb}} \leq \frac{0,36 I_1 E}{Z_p F_{ymin}} \sqrt{\frac{A_g J}{(I_1 - I_2)(I_1 - J/2,6)}} \tag{A.12.2-3}$$

where

I_1 is the major axis second moment of area of the gross cross-section;

I_2 is the minor axis second moment of area of the gross cross-section;

L_b is the effective length of a beam-column between supports, i.e. the length between points that are either braced against lateral displacement of the compression flange, or braced against twist of the cross-section, in addition to lateral support;

A_g is the gross cross-sectional area;

J is the torsion constant, $J = \dfrac{4A_o^2}{\sum(b_w / t)}$

where

A_o is the area enclosed by the median line of the perimeter material of the section,

b_w is the width of each component (wall of the section) forming the closed perimeter,

t is the thickness of each component (wall of the section) forming the closed perimeter;

r_{ltb} is the radius of gyration about the minor axis as defined in Formula (A.12.3-6);

$F_{y,ltb}$ is the yield strength, F_y of the material that first yields when bending about the minor axis. Conservatively, F_y in Formulae (A.12.2-2) and (A.12.2-3) may be taken as the maximum yield strength of all the components in a non-circular prismatic cross-section;

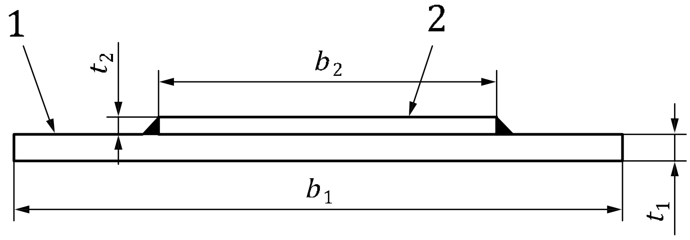
E is Young's modulus of steel;

Z_p is the fully plastic effective section modulus about the major axis determined from Formula (A.12.3-2);

F_{ymin} is the minimum yield strength of the F_{yi} of all components in the cross-section of a non-circular prismatic member, in stress units, as defined in A.12.2.2.

A.12.2.3.3 Reinforced components

Reinforcement of member cross-sections is often of the form shown in Figure A.12.2-1.



Key

1 base plate

2 reinforcing plate

b_1 width of base plate

b_2 width of reinforcing plate

t_1 thickness of base plate

t_2 thickness of reinforcing plate

Figure A.12.2-1 — Definitions for reinforced plate

To be considered a reinforcing plate, the plate should nominally be in contact with the base plate across its full width and continuously welded to the base plate on all edges with adequate welds.

When a reinforcing component is used, there should be four independent checks of the cross-section classification in accordance with Tables A.12.2-2 to A.12.2-4:

a) the reinforcing plate (using t_2) over the width b_2, using buckling coefficient increased by a factor of 1,573 (see below in A.12.2.3.3);

b) the combined plate using t_{check} over width b_1; see Formula (A.12.2-4);

c) the base plate (using t_1) over the width b_2 using buckling coefficient increased by a factor of 1,573 (see below in A.12.2.3.3);

d) the base plate (using t_1) over the dimension of the unreinforced widths (conservatively taken as $b_1 - b_2$).

If any component in the cross-section is found to be slender (class 4), then the effective width of those components, used in determining the thickness of the effective combined plate, should be determined from Table A.12.3-1. If the combined plate is found to be slender (class 4), its effective width should be determined from Table A.12.3-1.

Because the reinforcing plate is welded to the base plate around all edges, their ability to buckle independently over the width b_2 is restricted. Therefore, the coefficients in Tables A.12.2-2, A.12.2-4, and A.12.3-1 may be increased by a factor of 1,573 for cases a) and c) to account for this limited buckling capability.

NOTE As an example, the first limit in Table A.12.3-1, $0{,}72 t_f \sqrt{(E/F_y)}$, can be increased to $1{,}13\ t_f \sqrt{(E/F_y)}$ as derived from $1{,}13 = 0{,}72 \times 1{,}573$.

The reinforcing plate should be classified as a compression flange internal component or web internal component in accordance with Tables A.12.2-2 and A.12.2-4 depending on the type of in-plane loading. The value of yield stress used is that of the reinforcing plate.

The composite section should be classified as a compression flange internal component, a web internal component or a compression flange outstand component in accordance with Tables A.12.2-2 to A.12.2-4 depending on the type of in-plane loading and support conditions. The value of thickness t_{check} for use with width b_1 in the formulae in Table A.12.2-2 and A.12.2-4 should be determined from Formula (A.12.2-4):

$$t_{check} = (t^3_{eff}\, t_1)^{1/4} \tag{A.12.2-4}$$

where

$$t_{eff} = (12\, I/b_1)^{1/3} \tag{A.12.2-5}$$

$$I = [b_1(t_1 + t_2)^3 - (b_1 - b_2)t_2^3]/3 - A_{eff}(t_1 + t_2 - y_1)^2 \tag{A.12.2-6}$$

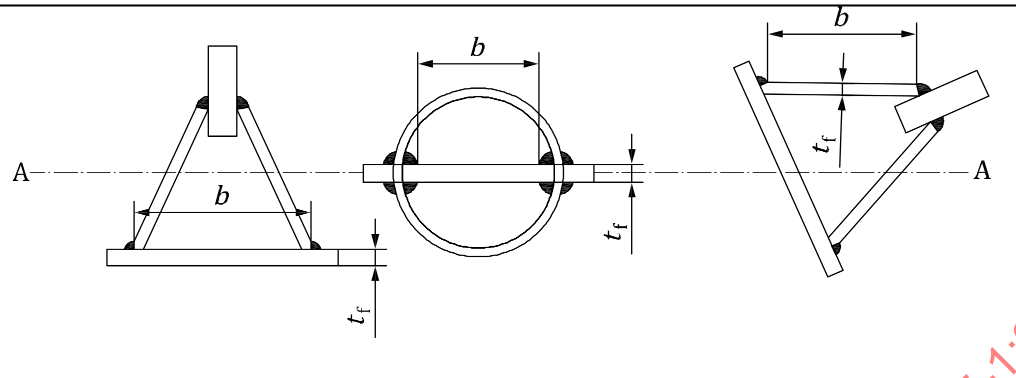
$$y_1 = [b_1 t_1^2 + b_2 t_2(2t_1 + t_2)]/(2A) \tag{A.12.2-7}$$

$$A_{eff} = b_1 t_1 + b_2 t_2 \tag{A.12.2-8}$$

The value of yield stress for use in Tables A.12.2-2 to A.12.2-4 is the larger of the yield stress values for the reinforcing plate or the base plate.

Table A.12.2-2 — Cross-section classification — Flange internal components

Limiting width-to-thickness ratios for compressed internal components



A-A is the axis of bending

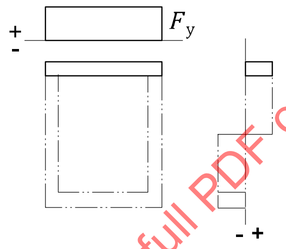
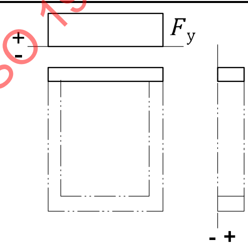
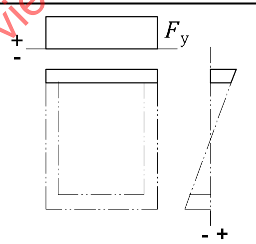
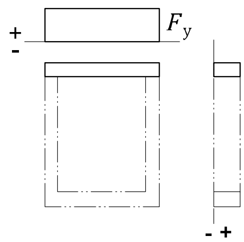
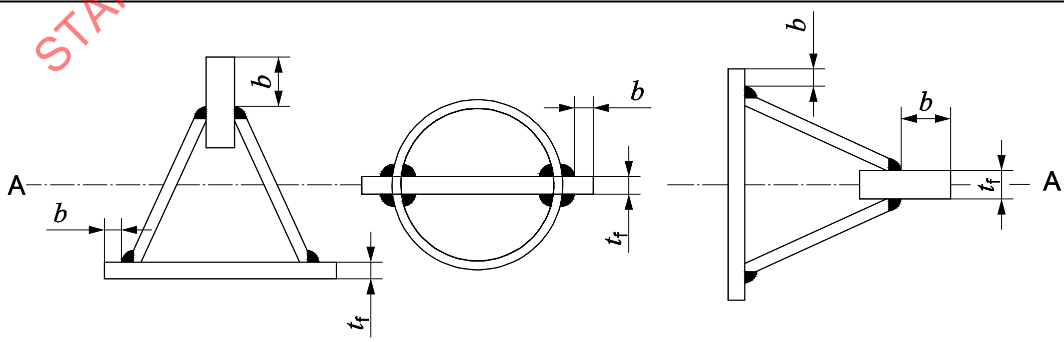
Class	Type	Section in bending	Section in compression
Plastic stress distribution in component and across section (compression positive)			
Plastic — Class 1	Rolled or welded	$b/t_f \le 1,03\sqrt{(E/F_y)}$	$b/t_f \le 1,03\sqrt{(E/F_y)}$
Compact — Class 2	Rolled or welded	$b/t_f \le 1,17\sqrt{(E/F_y)}$	$b/t_f \le 1,17\sqrt{(E/F_y)}$
Elastic stress distribution in component and across section (compression positive)			
Semi-Compact — Class 3	Rolled or welded	$b/t_f \le 1,44\sqrt{(E/F_y)}$	$b/_{tf} \le 1,44\sqrt{(E/F_y)}$
Slender — Class 4	Rolled or welded	$b/t_f > 1,44\sqrt{(E/F_y)}$	$b/t_f > 1,44\sqrt{(E/F_y)}$

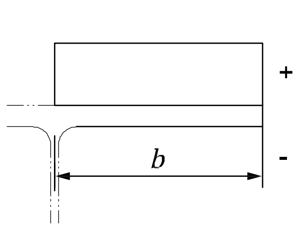
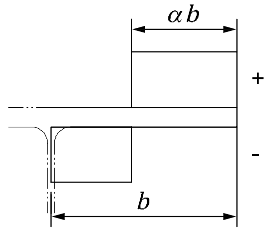
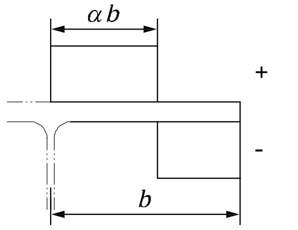
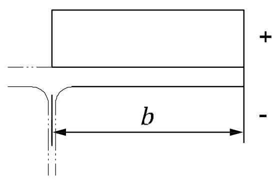
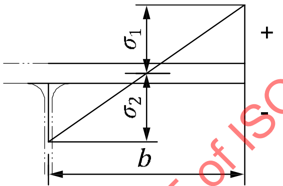
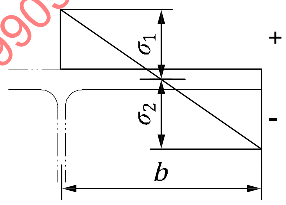
Table A.12.2-3 — Cross-section classification — Outstand components

Limiting width-to-thickness ratios for outstand components



A-A is the axis of bending

Class	Type	Outstand subject to compression	Outstand subject to compression and bending	
			Tip in compression	Tip in tension

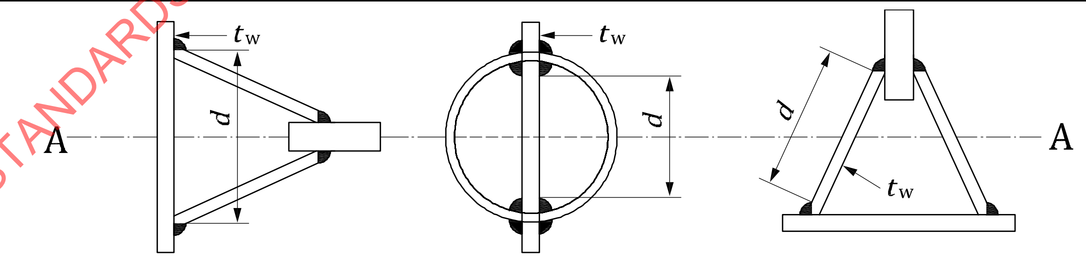
Plastic stress distribution in component (compression positive)				
Plastic — Class 1	Rolled Welded	$b/t_f \leq 0{,}33\surd(E/F_y)$ $b/t_f \leq 0{,}30\surd(E/F_y)$	$b/t_f \leq (0{,}33/\alpha)\surd(E/F_y)$ $b/t_f \leq (0{,}30/\alpha)\surd(E/F_y)$	$b/t_f \leq [0{,}33/(\alpha\surd\alpha)]\surd(E/F_y)$ $b/t_f \leq [0{,}30/(\alpha\surd\alpha)]\surd(E/F_y)$
Compact — Class 2	Rolled Welded	$b/t_f \leq 0{,}37\surd(E/F_y)$ $b/t_f \leq 0{,}33\surd(E/F_y)$	$b/t_f \leq (0{,}37/\alpha)\surd(E/F_y)$ $b/t_f \leq (0{,}33/\alpha)\surd(E/F_y)$	$b/t_f \leq [0{,}37/(\alpha\surd\alpha)]\surd(E/F_y)$ $b/t_f \leq [0{,}33/(\alpha\surd\alpha)]\surd(E/F_y)$
Elastic stress distribution in component (compression positive)			**Maximum compression at tip** 	**Maximum compression at connected edge** 
Semi-Compact — Class 3	Rolled Welded	$b/t_f \leq 0{,}55\surd(E/F_y)$ $b/t_f \leq 0{,}50\surd(E/F_y)$	$b/t_f \leq 0{,}84\surd(k_\sigma E/F_y)$ $b/t_f \leq 0{,}76\surd(k_\sigma E/F_y)$ $\psi = \sigma_2/\sigma_1$ $k_\sigma = 0{,}57 - 0{,}21\psi + 0{,}07\psi^2$ for $1 \geq \psi \geq -1$	$b/t_f \leq 0{,}84\surd(k_\sigma E/F_y)$ $b/t_f \leq 0{,}76\surd(k_\sigma E/F_y)$ $\psi = \sigma_2/\sigma_1$ $k_\sigma = 0{,}578/(\psi + 0{,}34$ for $1 \geq \psi \geq 0$ $k_\sigma = 1{,}7 - 5\psi + 17{,}1\psi^2$ for $0 > \psi \geq -1$
Slender — Class 4	Rolled or Welded	$b/t_f >$ than for Class 3	$b/t_f >$ than for Class 3	$b/t_f >$ than for Class 3

In the figures relating to stress distributions, the dimension, b, is illustrated only in the case of rolled sections. For welded sections, b should be assigned as shown in the diagrams at the top of the table.

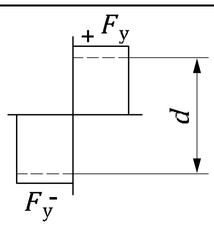
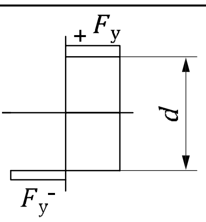
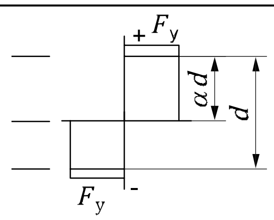
When determining α for Class 1 and 2 members, the loads should be scaled to give a fully plastic stress distribution. For all classes, it is conservative to use the relevant compression case.

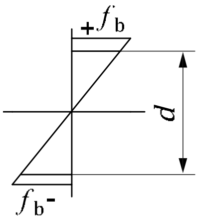
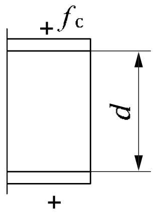
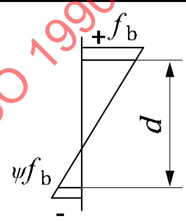
Table A.12.2-4 — Cross-section classification — Web internal components

Limiting width-to-thickness ratios for web internal components



A-A is the axis of bending

Class	Web subject to bending	Web subject to compression	Web subject to bending and compression
Plastic stress distribution in component (compression positive)			

Plastic — Class 1	$\alpha = 0{,}5$ $d/t_w \leq 2{,}56\sqrt{(E/F_y)}$	$\alpha = 1{,}0$ $d/t_w \leq 1{,}03\sqrt{(E/F_y)}$	when $\alpha > 0{,}5$ $d/t_w \leq \dfrac{5{,}18\sqrt{(E/F_y)}}{(6{,}043\,\alpha - 1)}$ when $\alpha \leq 0{,}5$ $d/t_w \leq 1{,}28\sqrt{(E/F_y)}/\alpha$
Compact — Class 2	$d/t_w \leq 3{,}09\sqrt{(E/F_y)}$	$d/t_w \leq 1{,}17\sqrt{(E/F_y)}$	when $\alpha > 0{,}5$ $d/t_w \leq \dfrac{4{,}82\sqrt{(E/F_y)}}{(5{,}12\,\alpha - 1)}$ when $\alpha \leq 0{,}5$ $d/t_w \leq \dfrac{1{,}55\sqrt{(E/F_y)}}{\alpha}$
Elastic stress distribution in component (compression positive)			
Semi-Compact — Class 3	$d/t_w \leq 4{,}14\sqrt{(E/F_y)}$	$d/t_w \leq 1{,}44\sqrt{(E/F_y)}$	when $\psi > -1{,}0$ $d/t_w \leq \dfrac{1{,}44\sqrt{(E/F_y)}}{(6{,}674 + 0{,}327\psi)}$ when $\psi \leq -1{,}0$ $d/t_w \leq 2{,}07(1-\psi)\sqrt{(-\psi)}\sqrt{(E/F_y)}$
Slender — Class 4	d/t_w > than for Class 3	d/t_w > than for Class 3	d/t_w > than for Class 3
When determining α for Class 1 and 2 members, the loads should be scaled to give a fully plastic stress distribution. For all classes it is conservative to use the relevant compression case.			

A.12.3 Section properties of non-circular prismatic members

A.12.3.1 General

Cross-sectional properties appropriate for the strength assessment of non-circular prismatic members of all classes should be determined as described in A.12.3.2 to A.12.3.4; the nomenclature and definition of variables is summarized in A.12.3.5. The properties appropriate for the stiffness assessment of prismatic members should be based on elastic considerations.

Where elastic section properties are determined for class 1 and class 2 sections in place of plastic section properties (e.g. for Euler amplification calculations or structural analysis input of stiffness parameters), these should be determined in accordance with A.12.3.3.

Cross-sectional properties are normally required in respect of both major and minor axes of a non-circular prismatic member.

Cross-sectional properties for tubular members are specified in A.12.5.

The cross-sectional properties used in the stiffness model (e.g. when determining structural deflections and natural periods) can differ from those used when assessing member strengths. For example, leg chord properties may include 10 % of the maximum rack tooth area when determining the leg stiffness. This additional material should not be included when calculating the section properties for strength assessment, except it may be used when determining the column buckling strength (see A.12.6.2.4) and moment amplification (see A.12.4).

A.12.3.2 Plastic and compact sections

A.12.3.2.1 Axial properties — Class 1 and class 2 sections

For class 1 plastic and class 2 compact sections, section properties should be determined assuming that fully plastic behaviour can occur. The properties required for a strength assessment should be determined taking into account the physical distribution of components comprising the cross-section and their yield strengths. For simplicity, the following approximations can be used to determine the relevant properties.

For axial tension and compression, the fully plastic effective cross-sectional area for use in a strength assessment, A_p is as given in Formula (A.12.3-1):

$$A_p = (\Sigma F_{yi} A_i)/F_{ymin} \quad \text{(A.12.3-1)}$$

where

F_{yi} is the yield strength of the ith component of the cross-section of a prismatic member, as defined in A.12.2.2;

A_i is the cross-sectional area of the ith component comprising the structural member;

F_{ymin} is the minimum yield strength of the F_{yi} of all components in the cross-section of a prismatic member, in stress units as defined in A.12.2.2.

NOTE 1 The centroid of the plastic section (or squash centre) of a member comprising components of differing yield strength can be offset from the centroid of the elastic section.

NOTE 2 A_p can be larger than the physical cross-section of the member.

A.12.3.2.2 Flexural properties — Class 1 and class 2 sections

The second moment of area, I_f, should be determined using the fully effective cross-section.

The fully plastic effective section modulus Z_p is as given in Formula (A.12.3-2):

$$Z_p = (\Sigma F_{yi} d_i A_i)/F_{ymin} \quad \text{(A.12.3-2)}$$

where d_i is the distance between the centroid of the ith component and the plastic neutral axis.

NOTE The plastic neutral axis does not necessarily coincide with the equal area axis for cross-sections composed of different yield strengths.

When using this definition of Z_p, the value of yield stress that should be used in the calculation of plastic moment strengths should be F_{ymin} as defined in A.12.2.2.

A.12.3.3 Semi-compact sections

For class 3 semi-compact sections, the section properties should be based on elastic properties assuming that the full cross-section is effective. The relevant variables are the cross-sectional area, A_f, as given in Formula (A.12.3-3), the second moment of area, I_f, and the elastic section modulus, S_f.

$$A_f = \Sigma A_i \quad \text{(A.12.3-3)}$$

The properties I_f and S_f should be determined assuming that the full cross-section is effective for bending about both major and minor axes. When considering a cross-section comprised of components having different yield strengths, the section moduli used in the calculations should encompass all critical points on the cross-section.

NOTE Critical stress locations are typically those at the edges of components and are a function of the member forces, the yield strength of the component and its position within the cross-section of the member.

A.12.3.4 Slender sections

A.12.3.4.1 General

Class 4 classification is determined from Tables A.12.2-1 to A.12.2-4. Cross-sectional properties for class 4 slender sections should be determined using elastic principles. In tension, fully effective sections should be assumed, i.e. A_f and S_f. In compression, the sectional properties should be based on effective sections as described here.

When analysing structures that contain class 4 sections, care should be taken when determining the force distributions. It is recommended that the structural analysis be performed using full elastic section properties and that the reduced section properties are used only for the member strength checks. Since this overestimates the forces in class 4 members, care should be taken when the use of the reduced sections causes a significantly different force distribution. In this case, an iterative analysis process can be required.

Effective sections should be based on actual plating thicknesses combined with plating effective widths. The effective widths of compression flange internal or outstand components should be determined in accordance with the formulae presented in Table A.12.3-1 a) or b), respectively. The effective widths of web internal components subject to compression and/or bending should be determined as shown in Table A.12.3-1 c) for which the following definitions apply (compression is taken as positive and tension as negative):

ψ is the ratio of compressive stress to bending stress;

σ_1 is the compressive stress if σ_2 is tensile or the larger compressive stress if σ_2 is also compressive;

σ_2 is the tensile stress if σ_2 is tensile or the smaller compressive stress if σ_2 is compressive;

k is the buckling coefficient (used in the assessment of the effective width of each components in a cross section);

ρ is the reduction coefficient;

λ_p is the plate slenderness parameter;

λ_{plim} is the limiting plate slenderness ratio;

λ_{po} is the plate slenderness ratio coefficient.

When determining effective widths for web internal components, the stress ratio, ψ, used in Table A.12.3-1 should be based on compression flange internal and outstand component effective widths, but the gross web section properties may be used.

The area reduction of curved components should be determined through the use of A.12.5.2.4. The following steps should be followed.

a) The representative local buckling strength should be determined for a tubular member with the wall thickness and diameter equivalent to the curved component in the non-circular prismatic member.

b) The strength of a tubular, of the same diameter and wall thickness used in step a), should be determined based on its full cross section and material yield.

c) The ratio of the strengths should be determined as in step a) strength divided by step b) strength.

d) This ratio of strengths should then be used to determine an equivalent reduced area of the curved component in the non-circular prismatic member.

The use of plating effective widths generally leads to a shift in the neutral axis compared with that found using gross sectional properties. This shift should be taken into account when determining effective widths. When the structural analysis is performed using gross section properties, the additional moment caused by the shift in the neutral axis should be found as the product of the axial force acting on the member and the shift in the neutral axis. This moment should be treated as additional to other moments acting on the effective section unless more onerous conditions arise if it is omitted.

A.12.3.4.2 Effective areas for compressive loading

The effective area $A_{eff,i}$ of a compressed component i should be found as the product of its thickness and its effective width (which should never be taken as greater than the actual width). The effective area of a curved component subject to uniform compression should be determined from its actual area reduced by the ratio of its strength when treated as a class 3 or class 4 tubular [Formula (A.12.5-8), when $0{,}170 < AF_y/P_{xe}$] versus its strength when treated as class 1 or class 2 tubular [Formula (A.12.5-8), when $AF_y/P_{xe} \leq 0{,}170$], as set out in steps a) to d) in A.12.3.4.1. The total effective area, A_{ec}, is the sum of the component effective areas, as given in Formula (A.12.3-4):

$$A_{ec} = \Sigma A_{eff,i} \quad \text{(A.12.3-4)}$$

A.12.3.4.3 Effective moduli for flexural loading

For web or flange internal components subject to combinations of flexural and compression loading, effective widths should be determined from Table A.12.3-1 c). For web or flange outstand components subject to combinations of flexural and compression loading, effective widths (which should never be taken as greater than the actual widths) should be determined from Table A.12.3-1 b). The effective area of a curved component subject to flexure should be determined from its actual area reduced by the ratio of its strength when treated as class 3 or class 4 tubular [Formula (A.12.5-13), for $0{,}103\,4 < (F_y D)/(E t)$] versus its strength when treated as class 1 tubular [Formula (A.12.5-13), for $(F_y D)/(E t) \leq 0{,}051\,7$], as set out in steps a) to d) in A.12.3.4.1. The effective second moment of area I_e should be found by calculating the properties of the section based on fully effective areas for components subject to tension, on effective areas as defined in A.12.3.4.2 for components subject to compression, and on effective areas as defined in the first paragraph for components subject to combinations of compression and flexure.

Application of this procedure to determine effective second moments of area when applied to cross-sections with slender components, especially when the section is not symmetric with respect to a particular axis, leads to two values of I_e about such an axis, depending upon the sign of the bending moment. Conservatively, the smaller value of I_e can be used throughout the strength analysis.

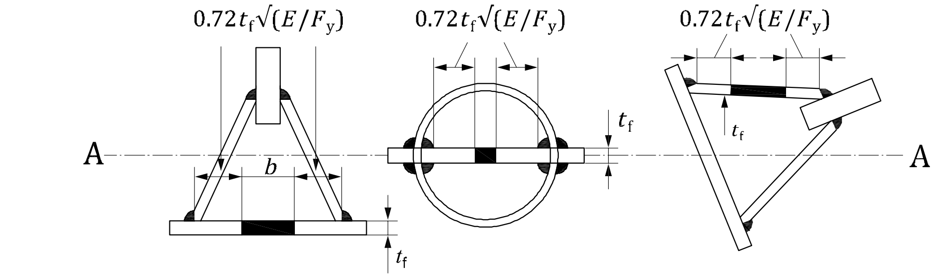
When considering a cross-section comprised of components having different yield strengths, the reduced elastic section modulus S_e used in the calculations should encompass all critical points on the cross-section:

$$S_e = I_e / y_i \tag{A.12.3-5}$$

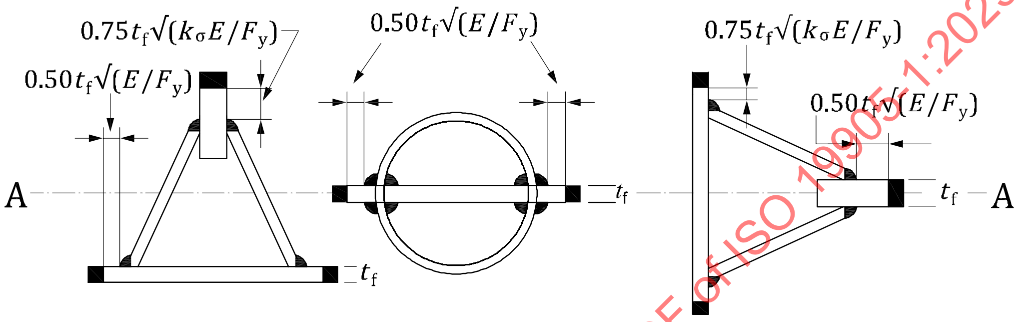
where y_i is the distance from the neutral axis associated with I_e to the critical point i.

NOTE Critical stress locations are typically those at the edges of components and are a function of the member forces, the yield strength of the component and its position within the cross-section of the member.

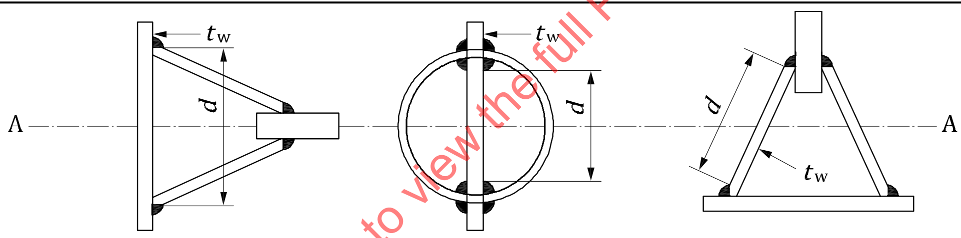
Table A.12.3-1 — Section properties — Effective widths for components in slender sections

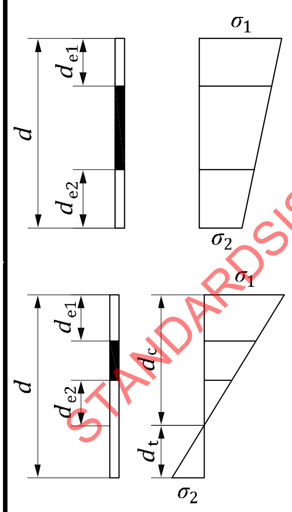


a) Compression internal components



b) Outstand components under compression and/or bending





$\psi = \sigma_2/\sigma_1$

$\lambda_p = 1{,}04(d/t_w)\sqrt{[F_y/(Ek)]}$

$1 \geq \psi \geq 0$	$\rho = 1$	if $\lambda_p \leq 0{,}75$
$d_{eff} = \rho\, d$	$\rho = [\lambda_p - 0{,}047(3+\psi)]/\lambda_p^2 \leq 1{,}0$	if $\lambda_p > 0{,}75$ and $(3+\psi) \geq 0$
$d_{e1} = 2d_{eff}/(5-\psi)$		
$d_{e2} = d_{eff} - d_{e1}$	$k = 8{,}2/(1{,}05+\psi)$	if $1 \geq \psi > 0$
	$k = 7{,}81 - 6{,}29\,\psi + 9{,}78\,\psi^2$	if $0 \geq \psi > -1$
	$k = 5{,}98\,(1-\psi)^2$	if $-1 \geq \psi > -3$
$\psi \leq 0$		
$d_{eff} = \rho\, d_c = \rho\, d/(1-\psi)$		
$d_{e1} = 0{,}4\, d_{eff}$		
$d_{e2} = 0{,}6\, d_{eff}$		

c) Internal components under compression and/or bending

Key

A-A axis of bending

 ineffective area, which is ignored when calculating effective section properties

NOTE 1 a) is a special case of c) with no bending and is included for clarity.

NOTE 2 The formula for K_σ, used in b), is given in Table A.12.2-3 in the row for “Semi-compact – Class 3” members.

A.12.3.5 Cross-sectional properties for the assessment

A.12.3.5.1 Tension

In tension, the cross-sectional area for use in the assessment should be A_t where

A_t $= A_p$ for class 1 plastic or class 2 compact sections, see Formula (A.12.3-1);

$= A_f$ for class 3 semi-compact sections as defined in Formula (A.12.3-3);

$= A_f$ as defined in Formula (A.12.3-3) for class 4 slender sections in tension across the whole of the cross-section (including bending); otherwise use A_{ec} for class 4 sections as defined by Formula (A.12.3-4).

Where the cross-section contains cut-outs, pin-holes, etc., A_t should be determined at the location of the minimum cross-section, unless the section is equipped with doubler plates surrounding the hole that at least replace all the lost area.

A.12.3.5.2 Compression

In compression, the cross-sectional area for use in the assessment should be A_c where

A_c $= A_p$ for class 1 plastic or class 2 compact sections, see Formula (A.12.3-1);

$= A_f$ for class 3 semi-compact sections as defined in Formula (A.12.3-3);

$= A_{ec}$ for class 4 slender sections as defined in Formula (A.12.3-4).

A.12.3.5.3 Flexure

In flexure, the second moment of area with respect to the y and z axes of bending that should be used in the assessment should be determined from the following:

$I_y, I_z = I_f$ for class 1 plastic and class 2 compact sections as defined in A.12.3.2.2;

$= I_f$ for class 3 semi-compact sections as defined in A.12.3.3;

$= I_e$ for class 4 slender sections as described in A.12.3.4.3 accounting for both the chosen axis and the direction of bending.

The section moduli for the two bending axes should be determined from the following:

S_y, S_z $= Z_p$ for class 1 plastic or class 2 compact sections, see Formula (A.12.3-2);

$= S_f$ for class 3 semi-compact sections as defined in A.12.3.3 for each critical stress location;

$= S_e$ for class 4 slender sections as defined in A.12.3.4.3 for each critical stress location, accounting for both the chosen axis and the direction of bending.

The radius of gyration about the minor axis that should be used for lateral-torsional buckling considerations, r_{ltb}, should be determined as given in Formula (A.12.3-6):

r_{ltb} $= (I_f / A_c)^{0,5}$ for sections in classes 1 to 3; (A.12.3-6)

$= (I_e / A_{ec})^{0,5}$ for sections in class 4.

A.12.4 Effects of axial force on bending moment

A.12.4.1 General

Euler moment amplification (p-δ) applies to all members in axial compression.

For classes 1, 2, and 3 cross-sections, the eccentricity between the elastic and plastic centroids induces an additional moment. This affects members in both tension and compression.

For class 4 members, in addition to the Euler moment amplification, there is an eccentricity between the full cross-section area normally used in the structural analysis and the effective neutral axis used in the member strength check. This can affect members in both tension and compression.

A.12.4.2 Member moment correction due to eccentricity of axial force

The plastic centroid or "centre of squash" is defined as the location at which the axial force produces no moment on the fully plastic section. For chords with material asymmetry (e.g. when the section includes components of differing yield strengths) the centre of squash can be offset from the elastic centroid. Before a section is checked, the moments should be corrected by the moment due to the axial force times the eccentricity between the elastic centroid (used in the structural analysis) and the "centre of squash" in accordance with Formula (A.12.4-1). There is no eccentricity for tubular members or for non-circular prismatic members with material symmetry.

The corrected effective moment, M_{ue}, should be calculated for each axis of bending, as given in Formula (A.12.4-1):

$$M_{ue} = M_u + eP_u \tag{A.12.4-1}$$

where

M_u is the moment in a member due to factored actions determined in an analysis that includes global P-Δ effects;

P_u is the axial force in the member due to factored actions determined in an analysis that includes global P-Δ effects;

e is the eccentricity between the axis used for structural analysis and that used for structural strength checks, taking due account of the sign in combination with the sign convention for P_u:

- e for class 1 and 2 members, is the distance between the elastic and plastic neutral axes orthogonal to the axis of bending under consideration. Annex F presents data including this offset distance (together with other geometric data) for many members of each chord family,
- e for class 3 members, is equal to e_a as defined in A.12.6.2.3,
- e for class 4 members, is the distance between the neutral axes of the full and effective cross-sections, orthogonal to the axis of bending under consideration,
- e is equal to 0 if the structural model fully accounts for the offset between the neutral axes of the modelled member in the strength checks,
- e is equal to 0 for tubular members; for other cross-sections in classes 1, 2 and 3 with material symmetry and when an elastic strength check is used for the assessment of members in classes 1, 2 and 3.

A.12.4.3 Member moment amplification and effective lengths

The amplified moment, M_{ua}, should be calculated for each axis of bending as given in Formula (A.12.4-2):

$$M_{ua} = B_{maf} M_{ue} \tag{A.12.4-2}$$

where

M_{ue} is as defined in A.12.4.2;

B_{maf} is the member moment amplification factor for the axis under consideration, equal to one of the following:

— $B_{maf} = 1{,}0$ for (i) members in tension, or (ii) members in compression where the individual member forces are determined from a second order analysis, i.e. the equilibrium conditions are formulated on the elastically deformed structure so that local p-δ effects are already included in M_u;

— $B_{maf} = \frac{C_{mr}}{(1-P_u/P_E)}$ for members in compression where the local member forces are determined from a first-order linear elastic analysis, i.e. the equilibrium conditions are formulated on the undeformed structure and therefore M_u does not include the local member p-δ effects:

where

$P_E = (\pi^2 E I)/(K L_{ub})^2$, and should be calculated for the plane of bending;

I	is the second moment of area for the plane of bending as defined in A.12.3.5.3 (including percentage of rack teeth of chords, see A.12.3.1);
K and C_{mr}	are given in Table A.12.4-1;
K	is the effective length factor for the plane of flexural buckling;
L_{ub}	is the unbraced length of member for the plane of flexural buckling normally taken as one of the following:

— face to face length for braces;

— braced point to braced point length for chords;

— longer segment length of X-braces (one pair is in tension, if not braced out-of-plane).

When the analysis of a jack-up with single-column tubular or box section legs has been undertaken accounting for the member moment amplification effects of global P-Δ/hull-sway, B_{maf} may be taken as 1,0 as local p-δ and global P-Δ are the same. For these jack-ups, local strength due to guide reactions should be assessed in conjunction with the member forces.

Table A.12.4-1 — Effective length and moment reduction factors

Structural member			K	C_{mr} [a]
Tubular or box complete legs			2,0[b]	A
Chords with lateral loading			1,0	C
Chords without lateral loading			1,0	B
Tubular braces	Primary diagonals and horizontals		0,7	B or C
	K-braces[c]		0,7	C
	X-brace[c]	Longer segment length	0,8	C
		Full length[d]	0,7	C
	Secondary horizontals (e.g. span-breakers)		0,7	B or C

a The value of C_{mr} can be determined from rational analysis. In lieu of such analysis, the following values may be used:

A For members whose ends are restrained against sidesway $C_{mr} = 0,85$

For members whose ends are unrestrained against sidesway $C_{mr} = 1,0$

B For members with no significant transverse loading, ignoring self-weight; buoyancy; and direct wave/current and wind actions:

$C_{mr} = 0,6 - 0,4\ M_1/M_2\ \leq 0,85$

where M_1/M_2 is the ratio of the smaller to the larger non-amplified end moments of the segment of the member in the plane of bending under consideration. M_1/M_2 is positive for the segment subject to reverse curvature and negative when subject to single curvature.

$M_1 = M_{ue}$ at end 1; similarly for M_2

C For members with significant transverse loading, other than distributed actions including self-weight; buoyancy; and direct wave/current and wind actions:

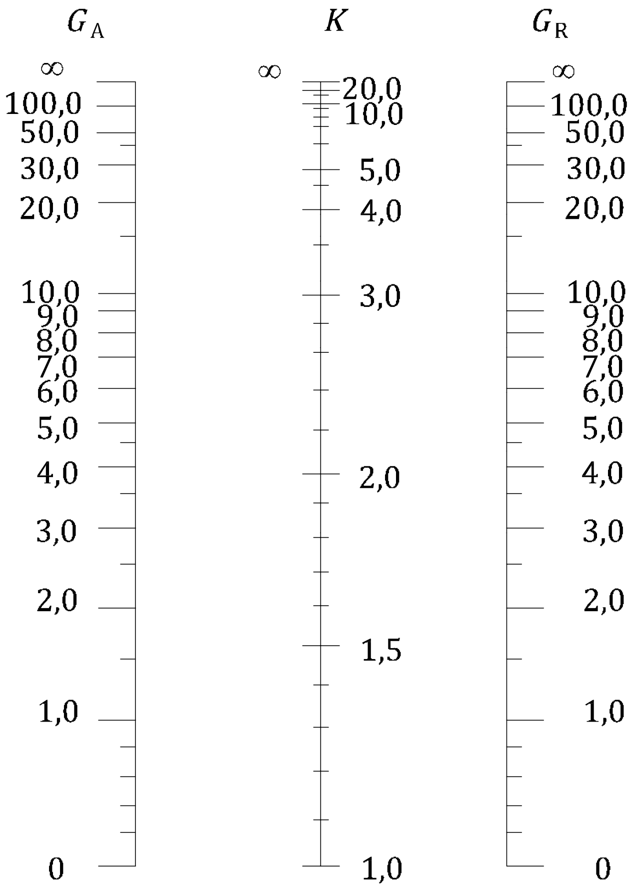
$C_{mr} = 1,0 - 0,2\ P_u/P_E\ \leq 0,85$ (AISC (2005)[13] Table C-C2.1, point actions)

$P_E = P_{Ey}$ or P_{Ez} as appropriate for the axis of bending under consideration.

b Alternatively use effective length alignment chart in Figure A.12.4-1.

c For either in-plane or out-of-plane effective lengths, at least one pair of members framing into a K- or X-joint is in tension if the joint is not braced out-of-plane.

d For X-braces, when all members are in compression and the joint is not braced out-of-plane.



To estimate the effective length of an unbraced column, such as tubular or box complete legs, the use of the alignment chart in Figure A.12.4-1 provides a simplified method for determining adequate K values. The alignment chart can be modified to allow for conditions different from those assumed in developing the chart.

The subscripts A and R refer to the joints at the two ends of the column section being considered. G is defined as

$$G = \frac{\Sigma \frac{I_c}{L_c}}{\Sigma \frac{I_0}{L_0}}$$

in which Σ indicates a summation of all members rigidly connected to that joint and lying in the plane in which buckling of the column is being considered. I_c is the moment of inertia and L_c the unsupported length of the column section, and I_0 is the moment of inertia and L_0 the unsupported length of a girder or other restraining member. I_c and I_0 are taken about axes perpendicular to the plane of buckling being considered.

For column ends supported by a pinned restraint, G is theoretically infinite but, unless truly friction free, can be taken as 10 for practical cases. If the column end is rigidly restrained, G may be taken as 1,0. Smaller values may be used if justified by analysis.

NOTE Taken from ISO 19902:2020, Figure A.13.5-4.

Figure A.12.4-1 — Alignment chart for determining the effective length of unbraced columns

A.12.5 Strength of tubular members

A.12.5.1 Applicability

The strength of unstiffened tubular members that satisfy Formula (A.12.5-1) should be assessed in accordance with A.12.5.

$$D/t \leq 0{,}2\ E/F_y \qquad \text{(A.12.5-1)}$$

Tubulars that do not satisfy Formula (A.12.5-1) should be assessed using alternative methods that result in levels of reliability comparable to those implicit in this document, such as DNV-RP-202 (DNV 2021b)[58] and API Bulletin 2U (2004)[16].

The strength formulae in A.12.5.1 to A.12.5.3 for $D/t \leq 0{,}2\ E/F_y$ are unconservative for tubulars with reductions in their cross-section. Where a tubular includes cross-sections with cut-outs, pin-holes, etc., it should be treated as for a non-circular prismatic member, unless it is adequately reinforced. Reinforcement can comprise either doubler plates that surround the hole or stiffeners that extend at least half the width of the hole above and below the hole. If the reinforcement replaces all the lost area of the tubular, the strength formulae in A.12.5 may be used.

The strength formulae are considered applicable for steels with a yield strength up to 800 N/mm². The yield strength used should be as specified in A.12.2.2.

NOTE The strength formulae for tubular members are based on ISO 19902:2020, Clause 13. However, for use in this document, the ISO 19902 formulations have been converted to a force base rather than a stress base. In addition, the combined axial, bending, beam shear and torsion checks have been simplified in A.12.5.

The formulae ignore the effect of hydrostatic pressure. The condition under which hydrostatic pressure can be ignored for a specific member is as given in Formula (A.12.5-2):

$$d_{w} \leq d_{w,lim} \quad \text{(A.12.5-2)}$$

where

d_{w} is the equivalent head of water in metres applicable to the tubular in question; it is the depth below the water surface (including penetration into the seabed where applicable) plus the additional soil pressure $p\gamma'/(\rho_{w}g)$;

$d_{w,lim}$ is the limiting head of water in metres applicable to the tubular in question, $d_{w,lim} = \left(\frac{211}{(D/t)}\right)^{2,985}$;

p is the depth below the sea floor in metres (zero if above sea floor);

γ' is the submerged (effective) unit weight of the soil;

ρ_{w} is the mass density of water;

g is the acceleration due to gravity.

For convenience, some typical maximum values of D/t, are given for a range of effective heads of water in Table A.12.5-1.

Table A.12.5-1 — Maximum $(D/t)_{m}$ values for given equivalent head of water

Equivalent head of water $d_{w,lim}$ m	Maximum tubular (D/t)
43	60,0
50	56,9
75	49,7
100	45,1
125	41,9
150	39,4
200	35,8

If the member D/t exceeds the maximum value of (D/t), the assessor should refer to ISO 19902, which is based on stress rather than strength.

A.12.5.2 Tension, compression and bending strength of tubular members

A.12.5.2.1 Yield strength to be used in calculating capacities

The presence of torsion can affect the axial, bending and shear capacities of tubular members. The flow of A.12.5 has been set up that all the basic member strength calculations include the effects of torsion, where appropriate. When the torsion is small, these effects reduce to zero impact. If there is a need to calculate simple axial and bending capacities without any torsional effects, then the actual yield strength of the material may be used in these checks to give an indicated, informative value. However, all reported strengths should include the effects of torsion.

The yield strength, $F_{y,T}$, which includes the effects of member torsion to be used in A.12.5 should be calculated using Formula (A.12.5-3):

$$F_{y,T} = F_y \qquad \text{when } T_u \leq \frac{0{,}2\, T_v}{\gamma_{R,Tv}}$$

$$F_{y,T} = \left(1 - \frac{T_u}{(T_v / \gamma_{R,Tv})}\right) F_y \qquad \text{when } T_u > \frac{0{,}2\, T_v}{\gamma_{R,Tv}} \qquad \text{(A.12.5-3)}$$

where

F_y is the yield strength in stress units as defined in A.12.2.2;

T_u is the torsional moment due to factored actions;

T_v is the representative torsional strength, calculated as given in Formula (A.12.5-4):

$$T_v = 2 I_{pt} F_y / (D \sqrt{3}) \qquad \text{(A.12.5-4)}$$

I_{pt} is the polar moment of inertia of a tube, calculated as given in Formula (A.12.5-5):

$$I_{pt} = (\pi/32)\, [D^4 - (D - 2t)^4] \qquad \text{(A.12.5-5)}$$

$\gamma_{R,Tv}$ is the partial resistance factor for torsional and beam shear strengths, $\gamma_{R,Tv} = 1{,}05$.

A.12.5.2.2 Axial tensile strength check

Tubular members subjected to axial tensile forces, P_{ut}, due to factored actions should satisfy Formula (A.12.5-6):

$$P_{ut} \leq A\, F_{y,T} / \gamma_{R,Tt} \qquad \text{(A.12.5-6)}$$

where

A is the gross cross-sectional area;

$\gamma_{R,Tt}$ is the partial resistance factor for axial tensile strength, $\gamma_{R,Tt} = 1{,}05$.

A.12.5.2.3 Axial compressive strength check

Tubular members subjected to axial compressive forces, P_{uc}, due to factored actions should satisfy Formula (A.12.5-7):

$$P_{uc} \leq P_a / \gamma_{R,Tc} \qquad \text{(A.12.5-7)}$$

where

P_a is the representative axial compressive strength as determined in A.12.5.2.5;

$\gamma_{R,Tc}$ is the partial resistance factor for axial compressive strength, $\gamma_{R,Tc} = 1{,}15$.

A.12.5.2.4 Local buckling strength

The representative local buckling strength, P_{yc}, should be determined as given in Formula (A.12.5-8):

$P_{yc} = A\,F_{y,T}$ (for $A\,F_{y,T}/P_{xe} \leq 0{,}170$)

$P_{yc} = (1{,}047 - 0{,}274\,A\,F_{y,T}/P_{xe})\,A\,F_{y,T}$ (for $0{,}170 < A\,F_{y,T}/P_{xe} \leq 0{,}333$) (A.12.5-8)

where, in addition to the variables in A.12.5.2.1, P_{xe} is the representative elastic local buckling strength, calculated as given in Formula (A.12.5-9):

$$P_{xe} = 2\,C_x\,E\,A\,(t/D) \quad \text{(A.12.5-9)}$$

where C_x is the critical elastic buckling coefficient.

The theoretical value of C_x for an ideal tubular is 0,6. However, a reduced value of $C_x = 0{,}3$ is recommended for use in the determination of P_{xe} to account for the effect of initial geometric imperfections. A reduced value of $C_x = 0{,}3$ is also implicit in the limits for $A\,F_{y,T}/P_{xe}$ given in Formula (A.12.5-8).

A.12.5.2.5 Column buckling strength

The representative axial compressive strength of tubular members, P_a, should be determined from Formulae (A.12.5-10) and (A.12.5-11):

$P_a = (1{,}0 - 0{,}278\lambda^2)P_{yc}$ (for $\lambda \leq 1{,}34$)

$P_a = 0{,}9\,P_{yc}/\lambda^2$ (for $\lambda > 1{,}34$) (A.12.5-10)

where

$$\lambda = (P_{yc}/P_E)^{0{,}5} \quad \text{(A.12.5-11)}$$

P_{yc} is the representative local buckling strength (see A.12.5.2.4);

λ is the column slenderness parameter;

P_E is the smaller of the Euler buckling strengths about the y- or z-direction, $P_E = \pi^2\,E\,I/(KL_{ub})^2$;

E is Young's modulus as defined in A.12.1.1;

K is the effective length factor in y- or z-direction; see A.12.4.3;

L_{ub} is the unbraced length in y- or z-direction; see A.12.4.3;

I is the second moment of area of the tubular.

A.12.5.2.6 Bending strength check

Tubular members subjected to bending moments, M_u, should satisfy Formula (A.12.5-12):

$$M_u \leq M_b\,/\gamma_{R,Tb} \quad \text{(A.12.5-12)}$$

where

M_u is equal to $\left(M_{uy}^2 + M_{uz}^2\right)^{0,5}$; it is the resolved bending moment due to factored actions about member y- and z-axes, respectively, determined in an analysis that includes global P-Δ effects;

M_b is the representative bending moment strength, determined as given in Formula (12.5-13):

$$M_b = M_p \quad \text{for } (F_{y,T}\, D)/(E\, t) \le 0{,}051\,7$$

$$M_b = [1{,}13 - 2{,}58\, (F_{y,T} D)/(E\, t)]\, M_p \quad \text{for } 0{,}051\,7 < (F_{y,T}\, D)/(E\, t) \le 0{,}103\,4 \qquad \text{(A.12.5-13)}$$

$$M_b = [0{,}94 - 0{,}76\, (F_{y,T} D)/(E\, t)]\, M_p \quad \text{for } 0{,}103\,4 < (F_{y,T}\, D)/(E\, t) \le 0{,}2$$

M_p is the plastic moment strength as given in Formula (A.12.5-14):

$$M_p = F_{y,T}\, [D^3 - (D - 2t)^3]/6 \qquad \text{(A.12.5-14)}$$

$\gamma_{R,Tb}$ is the partial resistance factor for bending strength, $\gamma_{R,Tb} = 1{,}05$.

A.12.5.2.7 Torsional shear strength check

Tubular members subjected to torsional shear forces due to factored actions should satisfy Formula (A.12.5-15):

$$T_u \le T_v\ /\gamma_{R,Tv} \qquad \text{(A.12.5-15)}$$

where

T_u is the torsional moment due to factored actions;

T_v is given in Formula (A.12.5-4).

A.12.5.2.8 Beam shear strength check

Tubular members subjected to beam shear forces due to factored actions should satisfy Formula (A.12.5-16):

$$V \le P_v/\gamma_{R,Tv} \qquad \text{(A.12.5-16)}$$

where

V is the beam shear due to factored actions;

P_v is the representative shear strength, as given in Formula (A.12.5-17):

$$P_v = A\, F_{y,T}/(2\sqrt{3}) \qquad \text{(A.12.5-17)}$$

NOTE P_v is calculated from $F_{y,T}$ according to Formula (A.12.5-3) and therefore accounts for the effects of torsion.

A is the gross cross-sectional area;

$\gamma_{R,Tv}$ is the partial resistance factor for torsional and beam shear strengths, $\gamma_{R,Tv} = 1{,}05$.

A.12.5.3 Tubular member combined strength checks

A.12.5.3.1 Axial tension and bending strength check

Tubular members subjected to combined axial tension and bending should satisfy the conditions given in Formulae (A.12.5-18) at all cross-sections along their length:

$$1-\cos\left(\frac{\pi}{2}\frac{\gamma_{R,Tt}P_{ut}}{AF_{y,T}}\right)+\frac{\gamma_{R,Tb}\left(M_{uy}^2+M_{uz}^2\right)^{0,5}}{M_b}\leq 1,0 \qquad \text{and} \qquad \gamma_{R,Tt}P_{ut}\leq AF_{y,T} \qquad \text{(A.12.5-18)}$$

where, in addition to the previously defined variables

P_{ut} is the axial tensile force due to factored actions;

A is the gross cross-sectional area;

$F_{y,T}$ is the yield strength in stress units as defined in A.12.5.2.1;

M_{uy}, M_{uz} are the bending moments due to factored actions about member y- and z-axes, respectively, determined in an analysis that includes global P-Δ;

M_b is the representative bending moment strength, as defined in Formula (A.12.5-13);

$\gamma_{R,Tt}$ is the partial resistance factor for axial tensile strength, $\gamma_{R,Tt} = 1{,}05$;

$\gamma_{R,Tb}$ is the partial resistance factor for bending strength, $\gamma_{R,Tb} = 1{,}05$.

NOTE If $\gamma_{R,Tt}P_{ut} > AF_{y,T}$ Formula (A.12.5-18) is not an appropriate measure of member utilization. A measure of the member utilization, U, can be obtained by calculating $U=\frac{\gamma_{R,Tt}P_{ut}}{AF_{y,T}}+\frac{\gamma_{R,Tb}\left(M_{uy}^2+M_{uz}^2\right)^{0,5}}{M_b}$

A.12.5.3.2 Axial compression and bending strength check

Tubular members subjected to combined axial compression and bending should satisfy the conditions given in Formulae (A.12.5-19) and (A.12.5-20) at all cross-sections along their length:

beam-column check:

$$(\gamma_{R,Tc}P_{uc}/P_a)+(\gamma_{R,Tb}/M_b)\,(M_{uay}{}^2+M_{uaz}{}^2)^{0,5}\leq 1{,}0 \qquad \text{(A.12.5-19)}$$

and local strength checks:

$$1-\cos\left(\frac{\pi}{2}\frac{\gamma_{R,Tc}P_{uc}}{P_{yc}}\right)+\frac{\gamma_{R,Tb}\left(M_{uy}^2+M_{uz}^2\right)^{0,5}}{M_b}\leq 1,0 \qquad \text{and} \qquad \gamma_{R,Tc}P_{uc}\leq P_{yc} \qquad \text{(A.12.5-20)}$$

where

P_{uc} is the axial compressive force due to factored actions;

P_{yc} is the representative local buckling strength in A.12.5.2.4;

P_a is the representative axial compressive strength as determined in A.12.5.2.5;

M_{uay} is the amplified bending moment about the member y-axis due to factored actions as determined in A.12.4.3;

M_{uaz} is the amplified bending moment about the member z-axis due to factored actions as determined in A.12.4.3;

M_b is the representative bending moment strength, as defined in Formula (A.12.5-13);

$\gamma_{R,Tb}$ is the partial resistance factor for bending strength, $\gamma_{R,Tb} = 1{,}05$;

$\gamma_{R,Tc}$ is the partial resistance factor for axial compressive strength, $\gamma_{R,Tc} = 1{,}15$.

NOTE If $\gamma_{R,Tc}P_{uc} > P_{yc}$ Equation A.12.5-20 is not an appropriate measure of member utilization. A measure of the member local strength utilization, U, can be obtained by calculating $U = \frac{\gamma_{R,Tc}P_{uc}}{P_{yc}} + \frac{\gamma_{R,Tb}\left(M_{uy}^2+M_{uz}^2\right)^{0,5}}{M_b}$.

A.12.5.3.3 Combined axial tension or compression, bending, shear and torsion strength check

The effect of shear on the strength of tubular members subjected to axial tension or compression or bending can be ignored if Formula (A.12.5-21), using the definitions of A.12.5.2.8, is satisfied:

$$V \leq \frac{0{,}7\,P_v}{\gamma_{R,Tv}} \tag{A.12.5-21}$$

If Formula (A.12.5-21) is not satisfied, reduced representative strengths should be calculated. The unreduced and reduced tensile strengths, $AF_{y,T,v}$, are given by Formula (A.12.5-22)

$$AF_{y,T,v} = AF_{y,T} \qquad \text{if } \frac{V}{\frac{P_v}{\gamma_{R,Tv}}} \leq 0{,}7$$

$$AF_{y,T,v} = \left[0{,}7 + 1{,}2\frac{V}{\frac{P_v}{\gamma_{R,Tv}}}\left(1 - \frac{V}{\frac{P_v}{\gamma_{R,Tv}}}\right)\right]AF_{y,T} \qquad \text{if } \frac{V}{\frac{P_v}{\gamma_{R,Tv}}} > 0{,}7 \tag{A.12.5-22}$$

the unreduced and reduced representative local buckling strengths, $P_{yc,v}$, are given by Formula (A.12.5-23):

$$P_{yc,v} = P_{yc} \qquad \text{if } \frac{V}{\frac{P_v}{\gamma_{R,Tv}}} \leq 0{,}7$$

$$P_{yc,v} = \left[0{,}7 + 1{,}2\frac{V}{\frac{P_v}{\gamma_{R,Tv}}}\left(1 - \frac{V}{\frac{P_v}{\gamma_{R,Tv}}}\right)\right]P_{yc} \qquad \text{if } \frac{V}{\frac{P_v}{\gamma_{R,Tv}}} > 0{,}7 \tag{A.12.5-23}$$

and the unreduced and reduced representative bending moment strengths, $M_{b,v}$, are given by Formula (A.12.5-24):

$$M_{b,v} = M_b \qquad \text{if } \frac{V}{\frac{P_v}{\gamma_{R,Tv}}} \leq 0{,}7$$

$$M_{b,v} = \left[0{,}7 + 1{,}2\frac{V}{\frac{P_v}{\gamma_{R,Tv}}}\left(1 - \frac{V}{\frac{P_v}{\gamma_{R,Tv}}}\right)\right]M_b \qquad \text{if } \frac{V}{\frac{P_v}{\gamma_{R,Tv}}} > 0{,}7 \tag{A.12.5-24}$$

NOTE 1 Formulae (A.12.5-22), (A.12.5-23) and (A.12.5-24) demonstrate that the reduced tensile strength, reduced local buckling strength and reduced bending strength equate to 70 % of their representative strengths even when fully utilized in shear.

Consequently, combinations of axial tension, bending and shear should satisfy Formula (A.12.5-25):

$$1 - \cos\left(\frac{\pi}{2}\frac{\gamma_{R,Tt}P_{ut}}{AF_{y,T,v}}\right) + \frac{\gamma_{R,Tb}\left(M_{uy}^2+M_{uz}^2\right)^{0,5}}{M_{b,v}} \leq 1{,}0 \quad \text{and} \quad \gamma_{R,Tt}P_{ut} \leq AF_{y,T,v} \tag{A.12.5-25}$$

NOTE 2 If $\gamma_{R,Tt}P_{ut} > AF_{y,T,v}$ Equation (A.12.5-25) is not an appropriate measure of member utilization. A measure of the member utilization, U, can be obtained by calculating $U = \frac{\gamma_{R,Tt}P_{ut}}{AF_{y,T,v}} + \frac{\gamma_{R,Tb}\left(M_{uy}^2+M_{uz}^2\right)^{0,5}}{M_{b,v}}$

Combinations of axial compression, bending and shear should satisfy Formulae (A.12.5-26) and (A.12.5-27):

$$\frac{\gamma_{R,Tc}P_{uc}}{P_a} + \frac{\gamma_{R,Tb}\left(M_{uay}^2+M_{uaz}^2\right)^{0,5}}{M_{b,v}} \leq 1,0 \tag{A.12.5-26}$$

$$1-\cos\left(\frac{\pi}{2}\frac{\gamma_{R,Tc}P_{uc}}{P_{yc,v}}\right) + \frac{\gamma_{R,Tb}\left(M_{uy}^2+M_{uz}^2\right)^{0,5}}{M_{b,v}} \leq 1,0 \quad \text{and} \quad \gamma_{R,Tc}P_{uc} \leq P_{yc} \tag{A.12.5-27}$$

In Formula (A.12.5-26), the axial compression strength, P_a, should be calculated from Formula (A.12.5-10) using $P_{yc,v}$ instead of P_{yc}.

NOTE 3 If $\gamma_{R,Tc}P_{uc} > P_{yc}$ then Formula (A.12.5-27) is not an appropriate measure of member utilization. A measure of the local strength member utilization, U, can be obtained by calculating $U = \frac{\gamma_{R,Tc}P_{uc}}{P_{yc}} + \frac{\gamma_{R,Tb}\left(M_{uy}^2+M_{uz}^2\right)^{0,5}}{M_{b,v}}$.

A.12.6 Strength of non-circular prismatic members

A.12.6.1 General

The structural strength provisions for rolled and welded non-circular prismatic members are generally based on the AISC (2005)[13]. The AISC (2005)[13] specification for LRFD was interpreted and, in some cases, modified for use in the assessment of mobile jack-up structures. The strength formulae for column buckling for lower strength steels in A.12.6.2.4 were modified for consistency with the approach used for higher strength steels, which was taken from Galambos (1998)[75]. Interpretation of the specifications was necessary to enable presentation of a straightforward method for the assessment of beam-columns with components of varying yield strength and/or with cross-sections having only a single axis of symmetry. Development of the specifications was necessary to provide the following:

a) a method to deal with member cross-sections comprising components constructed of steels with different yield strengths;

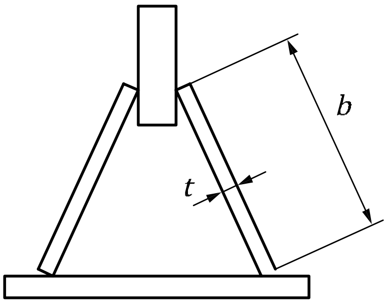
b) a method for the assessment of beam-columns under biaxial bending to overcome a conservatism that has been identified in the standard AISC interaction formulae.

The yield strength used in A.12.6 should be as specified in A.12.2.2.

The effects of hydrostatic loading on non-circular prismatic members should be considered. The critical condition for hydrostatic loading on non-circular prismatic chord members is likely to occur when high spudcan fixity results in high chord axial loads in deep water.

Hydrostatic pressure effects on split tubular and similar members should be addressed as described in A.12.5.1. If the section fails to meet the un-reinforced tubular check, additional analysis can be used to determine the effects of the stiffening provided by the non-tubular components.

Hydrostatic pressure effects on flat plate components of members should be assessed as shown in Figure A.12.6-1 for values of β less than 2,0. If the component is used under conditions with an equivalent head of water greater than the limiting equivalent head of water given in Figure A.12.6-1, or if the calculated β is greater than 2,0, then rational analysis should be used to assess the effects of hydrostatic pressure on member utilization. For convenience, Table A.12.6-1 gives the limiting equivalent head of water for components of differing plate slendernesses.



$d_{w,lim} = 298\beta^4 - 2\,092\beta^3 + 5\,542\beta^2 - 6\,603\beta + 3\,025$ for $\beta \le 2{,}0$

Key

b	width of base plate
t	thickness of base plate
$d_{w,lim}$	limiting equivalent head of water in metres for which additional analysis is not required; it is the depth below the water surface (including penetration into the seabed where applicable) plus the additional soil pressure $p\gamma'/(\rho_w g)$
β	plate slenderness parameter $\beta = (b/t)(F_y/E)^{0,5}$
p	is the depth below the sea floor in metres (zero if above sea floor)
γ'	is the submerged (effective) unit weight of the soil
ρ_w	is the mass density of water

Figure A.12.6-1 — Example chord showing plate dimensions for hydrostatic pressure screening check

Table A.12.6-1 — Maximum plate slenderness parameter β for given equivalent head of water

Equivalent head of water $d_{w,lim}$ m	Maximum plate slenderness parameter β
170	1,0
120	1,1
85	1,2
48	1,4
32	1,6
24	1,8
20	2,0

In A.12.6.2 and A.12.6.3, y and z are used to define the axes of a non-circular prismatic member.

A.12.6.2 Non-circular prismatic members subjected to tension, compression, bending or shear

A.12.6.2.1 General

Non-circular prismatic members subjected to axial tension, axial compression, bending or shear should satisfy the applicable strength and stability checks specified in A.12.6.2.2 to A.12.6.2.7.

A.12.6.2.2 Axial tensile strength check

Non-circular prismatic members subjected to axial tensile forces, P_{ut}, due to factored actions should satisfy Formula (A.12.6-1):

$$P_{ut} \le P_t/\gamma_{R,Pt} \tag{A.12.6-1}$$

where

P_t is the representative axial tensile strength of a non-circular prismatic member, calculated as given in Formula (A.12.6-2)

$$P_t = \Sigma(F_{yi}A_i) \quad \text{(A.12.6-2)}$$

F_{yi} is the yield strength of the ith component of the cross-section of a prismatic member, in stress units, as defined in A.12.2.2;

A_i is the cross-sectional area of the ith component comprising the structural member;

$\gamma_{R,Pt}$ is the partial resistance factor for axial tensile strength, $\gamma_{R,Pt} = 1{,}05$.

A.12.6.2.3 Axial compressive local strength check

Non-circular prismatic members subjected to axial compressive forces, P_{uc}, due to factored actions should satisfy Formula (A.12.6-3):

$$P_{uc} \leq P_{pl}/\gamma_{R,Pcl} \quad \text{(A.12.6-3)}$$

where, in addition to the definitions given in A.12.6.2.2

$\gamma_{R,Pcl}$ is the partial resistance factor for local axial compressive strength, $\gamma_{R,Pcl} = 1{,}1$;

P_{pl} is the representative local axial compressive strength of a non-circular prismatic member as given in Formulae (A.12.6-4) to (A.12.6-6);

$$P_{pl} = \Sigma F_{yi}A_i \quad \text{for class 1 and class 2 members} \quad \text{(A.12.6-4)}$$

$$P_{pl} = \Sigma F_{yi}A_i - (\Sigma F_{yi}A_i - F_{ymin}\Sigma A_i)\left(\frac{\lambda_h - \lambda_p}{\lambda_r - \lambda_p}\right)_h \quad \text{for class 3 members} \quad \text{(A.12.6-5)}$$

$$P_{pl} = F_{ymin}A_c \quad \text{for class 4 members} \quad \text{(A.12.6-6)}$$

F_{ymin} is the minimum yield stress of the F_{yi} of all components in the cross-section of a prismatic member, in stress units, as defined in A.12.2.2;

A_i is the cross-sectional area of the ith component comprising the structural member;

A_c is the cross-sectional area for use in the assessment of a non-circular prismatic member as defined in A.12.3.5.2;

h is the subscript referring to the component that produces the smallest value of P_{pl};

λ_h $= b/t$ or $2R/t$ as applicable for component h, with effective width b, or outside radius R;

λ_p is as determined for component h from Tables A.12.2-2 to A.12.2-4 as given in Formulae (A.12.6-7) to (A.12.6-10):

— for rectangular rolled or welded web or flange components supported along both edges:

$$\lambda_p = 1,17\sqrt{E / F_{yi}} \tag{A.12.6-7}$$

— for rectangular rolled flange or web components supported along one edge:

$$\lambda_p = 0,37\sqrt{E / F_{yi}} \tag{A.12.6-8}$$

— for rectangular welded flange or web components supported along one edge:

$$\lambda_p = 0,33\sqrt{E / F_{yi}} \tag{A.12.6-9}$$

— for components derived from tubulars (with reference to Table A.12.2-1):

$$\lambda_p = 0,077E / F_{yi} \tag{A.12.6-10}$$

λ_r is determined for component h from Tables A.12.2-2 to A.12.2-4 as given in Formulae (A.12.6-11) to A.12.6-14):

— for rectangular rolled or welded web or flange components supported along both edges:

$$\lambda_r = 1,44\sqrt{E / F_{yi}} \tag{A.12.6-11}$$

— for rectangular rolled flange or web components supported along one edge:

$$\lambda_r = 0,55\sqrt{E / F_{yi}} \tag{A.12.6-12}$$

— for rectangular welded flange or web components supported along one edge:

$$\lambda_r = 0,50\sqrt{E / F_{yi}} \tag{A.12.6-13}$$

— for components derived from tubulars (with reference to AISC (2005)[13] and Table A.12.2-1):

$$\lambda_r = 0,102E / F_{yi} \tag{A.12.6-14}$$

The eccentricity between the elastic and plastic neutral axes, e_a, for class 3 members (see A.12.4) can be calculated as given in Formula (A.12.6-15):

$$e_a = e\left(\frac{\lambda_r - \lambda_h}{\lambda_r - \lambda_p}\right)_h \tag{A.12.6-15}$$

where e is as defined in A.12.4.2, but calculated for the cross section as if it were class 1 or 2.

A.12.6.2.4 Axial compressive column buckling strength

There is no axial compressive column buckling strength check because it is inherent in the combined strength check for compression in A.12.6.3. However, the representative axial compressive strength of all member classifications subjected to flexural buckling should be determined as given Formulae (A.12.6-16) to (A.12.6-19):

a) for all grades of steel (conservative for high strength steel):

$$P_n = \left(0{,}658^{\lambda_c^2}\right) P_{pl} \quad \text{for } \lambda_c \le 1{,}5 \text{ [derived from AISC (2005)[13], Formula E3-2]} \qquad \text{(A.12.6-16)}$$

$$P_n = \left(0{,}877 / \lambda_c^2\right) P_{pl} \quad \text{for } \lambda_c > 1{,}5 \text{ [derived from AISC (2005)[13], Formula E3-3]} \qquad \text{(A.12.6--17)}$$

b) alternatively, for high-strength steels ($F_y > 450$ MPa), the following may be used (see F.1):

$$P_n = \left(0{,}762\,5^{\lambda_c^{3{,}22}}\right) P_{pl} \quad \text{for } \lambda_c \le 1{,}2 \qquad \text{(A.12.6-18)}$$

$$P_n = \left(0{,}860\,8 / \lambda_c^{1{,}854}\right) P_{pl} \quad \text{for } \lambda_c > 1{,}2 \qquad \text{(A.12.6-19)}$$

where, in addition to the definitions in A.12.6.2.3,

$$\lambda_c = \left(\frac{P_{pl}}{P_E}\right)^{0{,}5} \quad \text{(derived from AISC (2005)[13], Ch. E3; see also F.1)} \qquad \text{(A.12.6-20)}$$

P_E is the minimum Euler buckling load for any plane of bending, as defined in A.12.4.3 (including percentage of rack teeth of chords; see A.12.3.1).

When section contains un-reinforced cut-outs, the slenderness parameter, λ_c, should be based on the minimum section unless otherwise determined by analysis.

A.12.6.2.5 Bending moment strength

A.12.6.2.5.1 General

The classification of member cross-sections in A.12.2 is used to identify the potential for local buckling. The slender section properties determined in A.12.3.4 account for the local buckling of class 4 cross-sections.

The bending moment strength of typical closed section jack-up chord members used in truss legs is not normally limited by lateral torsional buckling. However, this should be checked as described in A.12.2.3.2.

A.12.6.2.5.2 Class 1 plastic and class 2 compact section bending moment strength

The representative bending moment strength, M_b, is given by the plastic bending moment of the entire section as given in Formula (A.12.6-21):

$$M_b = Z_p F_{ymin} \qquad \text{(A.12.6-21)}$$

where

M_b is the representative bending moment strength;

Z_p is the fully plastic effective section modulus, determined from Formula (A.12.3-2);

F_{ymin} is the minimum yield strength of all components in the cross-section of a prismatic member, in stress units, as defined in A.12.2.2.

NOTE Hybrid sections built up from components of different yield strengths are addressed by the methodology described in A.12.3.2.

A.12.6.2.5.3 Class 3 semi-compact section bending moment strength

The representative bending strength, M_b, is obtained by interpolating between the plastic bending moment and the limiting buckling moment as given in Formula (A.12.6-22):

$$M_b = M_p - \left(M_p - M_R\right)\left(\frac{\lambda_h - \lambda_p}{\lambda_r - \lambda_p}\right)_h \quad \text{(A.12.6-22)}$$

where, in addition to the definitions in A.12.6.2.5.2

M_p is the plastic moment strength;

$M_p = Z_p F_{ymin}$ as calculated by Formula (A.12.6-21);

$$M_R = S_f F_y < M_p \quad \text{(A.12.6-23)}$$

S_f is the elastic section modulus of a semi-compact section of a non-circular prismatic member for the plane of bending under consideration; see A.12.3.3;

F_y is the yield strength of the material at the critical point in the cross-section, determined when calculating the section modulus (see A.12.3.3);

h is the subscript referring to the component which produces the smallest value of M_b;

λ_h $= b/t$ or $2R/t$ as applicable for component h;

λ_p is as determined for component h from Tables A.12.2-2 to A.12.2-4, as given in Formulae (A.12.6-24) to (A.12.6-29):

— for rectangular rolled or welded flange components supported along both edges when the bending results in uniform compression:

$$\lambda_p = 1{,}17\sqrt{(E / F_{yi})} \quad \text{(A.12.6-24)}$$

— for rectangular rolled flange or web components supported along one edge and subject to combinations of compression and bending:

$$\lambda_p = 0{,}37\sqrt{(E / F_{yi})} \quad \text{(A.12.6-25)}$$

— for rectangular welded flange or web components supported along one edge and subject to combinations of compression and bending:

$$\lambda_p = 0{,}33\sqrt{(E / F_{yi})} \quad \text{(A.12.6-26)}$$

— for rectangular rolled or welded web components supported along both edges and subject to combinations of compression and bending:

$$\lambda_p = \left[4,82\sqrt{(E / F_{yi})}\right] / (5,12\alpha - 1) \qquad \text{(for } \alpha > 0,5) \qquad (A.12.6\text{-}27)$$

$$\lambda_p = \left[1,55\sqrt{(E / F_{yi})}\right] / \alpha \qquad \text{(for } \alpha \leq 0,5) \qquad (A.12.6\text{-}28)$$

where α is a factor that varies depending on the applied loading, given in Table A.12.2-4, and equals 0,5 in bending, 1,0 in compression, and variable between these values for combined bending and compression.

— for components derived from circular tubes and subject to pure bending (see Table A.12.2-1):

$$\lambda_p = 0,103E / F_{yi} \qquad (A.12.6\text{-}29)$$

When the location of the tubular component results in combined bending and compression the value of λ_p can conservatively be taken from Formula (A.12.6-10). Alternatively, the value of λ_p may be interpolated between the values for pure bending and pure compression.

λ_r is determined for component *h* from Tables A.12.2-2 to A.12.2-4, as given in Formulae (A.12.6-30) to (A.12.6-35):

— for rectangular rolled or welded flange components supported along both edges when the bending results in uniform compression:

$$\lambda_r = 1,44\sqrt{(E / F_{yi})} \qquad (A.12.6\text{-}30)$$

— for rectangular rolled flange or web components supported along one edge and subject to combinations of compression and bending:

$$\lambda_r = 0,55\sqrt{(E / F_{yi})} \qquad (A.12.6\text{-}31)$$

— for rectangular welded flange or web components supported along one edge and subject to combinations of compression and bending:

$$\lambda_r = 0,50\sqrt{(E / F_{yi})} \qquad (A.12.6\text{-}32)$$

— for rectangular rolled or welded web components supported along both edges and subject to combinations of compression and bending:

$$\lambda_r = \left[1,44\sqrt{(E / F_{yi})}\right] / \left(0,674 + 0,327\psi\right) \qquad \text{(for } \psi > -1,0) \qquad (A.12.6\text{-}33)$$

$$\lambda_r = \left[2,07\left(1 - \psi\right)\sqrt{\left(-\psi\right)}\right]\sqrt{\left(E / F_{yi}\right)} \qquad \text{(for } \psi \leq_i -1,0) \qquad (A.12.6\text{-}34)$$

where ψ is the stress ratio as shown in Table A.12.2-4.

— for components derived from circular tubes and subject to pure bending (see Table A.12.2-1):

$$\lambda_r = 0,22E / F_{yi} \quad \text{(A.12.6-35)}$$

When the location of the tubular component results in combined bending and compression, the value of λ_r can conservatively be taken from Formula (A.12.6-14). Alternatively, the value of λ_r may be interpolated between the values for pure bending and pure compression.

A.12.6.2.5.4 Class 4 slender-section bending moment strength

The representative bending moment strength, M_b, of class 4 sections is given by the limiting flexural bending moment in Formula (A.12.6-36):

$$M_b = S_e F_y \quad \text{(A.12.6-36)}$$

where S_e is the reduced elastic section modulus of a slender section of a non-circular prismatic member for the plane of bending under consideration, see A.12.3.4.3 and F_y is the yield strength of the material at the critical point in the cross-section, determined when calculating the section modulus (see A.12.3.3).

A.12.6.2.6 Bending moment strength affected by lateral torsional buckling

The reduced representative bending moment strength M_b due to LTB should be calculated for all members that do not meet the screening checks of either Formula (A.12.2-2) or Formula (A.12.2-3) for open and closed sections, respectively, regardless of the class of section. When the representative bending moment strength is reduced due to LTB compared to the strength calculated in A.12.6.2.5, the reduced bending moment strength should be used in the strength checks.

Further guidance on the bending moment strength accounting for LTB can be found in the AISC Specification (2005)[13] and BS 5400-3.

A.12.6.2.7 Bending strength check

Non-circular prismatic members subjected to bending moments, M_u, should satisfy Formula (A.12.6-37):

$$M_u \leq M_b / \gamma_{R,Pb} \quad \text{(A.12.6-37)}$$

where

M_u is M_{uy} or M_{uz}, the bending moment due to factored actions about member y- and z-axes, respectively;

M_b is the representative bending moment strength, determined from A.12.6.2.5 and A.12.6.2.6;

$\gamma_{R,Pb}$ is the partial resistance factor for bending, $\gamma_{R,Pb} = 1,1$.

A.12.6.3 Non-circular prismatic member combined strength checks

A.12.6.3.1 General

There are two different assessment approaches for the strength of non-circular prismatic members subjected to combined axial forces and bending moments:

a) the interaction formula approach (see A.12.6.3.2), which is applicable to all member classifications;

b) the plastic interaction surface approach (see A.12.6.3.3), which is applicable to members in class 1 and class 2.

A.12.6.3.2 Interaction formula approach

Each non-circular prismatic structural member should satisfy the following conditions in Formulae (A.12.6-38) to (A.12.6-40) at all cross-sections along its length. When the shear due to factored actions is greater than 60 % of the shear strength, the bending moment strength should be reduced parabolically to zero when the shear equals the shear strength (P_v in A.12.6.3.4).

Local strength check (for all members) is as given in Formula (A.12.6-38):

$$\frac{\gamma_{R,Pa} P_u}{P_{pls}} + \left[\left(\frac{\gamma_{R,Pb} M_{uey}}{M_{by}}\right)^{\eta} + \left(\frac{\gamma_{R,Pb} M_{uez}}{M_{bz}}\right)^{\eta}\right]^{\frac{1}{\eta}} \leq 1{,}0 \tag{A.12.6-38}$$

Beam-column check (for members subject to axial compression) is as given in Formula (A.12.6-39) or Formula (A.12.6-40):

— if $\gamma_{R,Pa} P_u / P_p > 0{,}2$, then (after AISC (2005)[13], Formula H1-1a)

$$\frac{\gamma_{R,Pa} P_u}{P_p} + \frac{8}{9}\left[\left(\frac{\gamma_{R,Pb} M_{uay}}{M_{by}}\right)^{\eta} + \left(\frac{\gamma_{R,Pb} M_{uaz}}{M_{bz}}\right)^{\eta}\right]^{\frac{1}{\eta}} \leq 1{,}0 \tag{A.12.6-39}$$

— if $\gamma_{R,Pa} P_u / P_p \leq 0{,}2$, then (after AISC (2005)[13], Formula H1-1b)

$$\frac{\gamma_{R,Pa} P_u}{2 P_p} + \left[\left(\frac{\gamma_{R,Pb} M_{uay}}{M_{by}}\right)^{\eta} + \left(\frac{\gamma_{R,Pb} M_{uaz}}{M_{bz}}\right)^{\eta}\right]^{\frac{1}{\eta}} \leq 1{,}0 \tag{A.12.6-40}$$

where

P_u is the applied axial force in a member due to factored actions, determined in an analysis that includes P-Δ effects (see A.12.4);

P_{pls} is the representative local axial strength of a non-circular prismatic member where

$P_{pls} = P_t$ for members in tension, as defined A.12.6.2.2,

$P_{pls} = P_{pl}$ for members in compression, as defined A.12.6.2.3;

P_p is the representative axial strength of a non-circular prismatic member where

$P_p = P_n$ for members in compression, as defined A.12.6.2.4;

M_{uey} is the corrected bending moment due to factored actions about the member y-axis from A.12.4;

M_{uez} is the corrected bending moment due to factored actions about the member z-axis from A.12.4;

M_{uay} is the amplified bending moment due to factored actions about the member y-axis from A.12.4;

M_{uaz} is the amplified bending moment due to factored actions about the member z-axis from A.12.4;

M_{by} is the representative bending moment strength about the member y-axis, as defined in A.12.6.2.5 or A.12.6.2.6;

When the shear due to factored actions is greater than 60 % of the shear strength, the bending moment strength should be reduced parabolically to zero when the shear equals the shear strength (P_{vz} in A.12.6.3.4). For a more detailed description of the method see EN 1993-1-1, *Eurocode 3*;

M_{bz} is the representative bending moment strength about the member z-axis, as defined in A.12.6.2.5 or A.12.6.2.6;

When the shear due to factored actions is greater than 60 % of the shear strength, the bending moment strength should be reduced parabolically to zero when the shear equals the shear strength (P_{vy} in A.12.6.3.4). For a more detailed description of the method see EN 1993-1-1, *Eurocode 3*;

$\gamma_{R,Pb}$ is the partial resistance factor for bending strength, $\gamma_{R,Pb} = 1{,}1$;

$\gamma_{R,Pa}$ is the partial resistance factor for axial strength where

$\gamma_{R,Pa} = \gamma_{R,Pt}$ for axial tensile strength, $\gamma_{R,Pa} = 1{,}05$ in Formulae (A.12.6-38), (A.12.6-39) and (A.12.6-40);

$\gamma_{R,Pa} = \gamma_{R,Pcl}$ for axial compressive strength, $\gamma_{R,Pa} = 1{,}1$ in Formula (A.12.6-38);

$\gamma_{R,Pa} = \gamma_{R,Pc}$ for axial compressive strength, $\gamma_{R,Pa} = 1{,}1$ in Formulae (A.12.6-39) and (A.12.6-40).

η is the exponent for biaxial bending, a constant dependent on the member cross-section geometry, determined as follows:

— for purely circular tubular members $\eta = 2{,}0$;

— for solid or hollow rectangular sections $\eta = 5/3$;

— for doubly symmetric open section members $\eta = 1{,}0$;

— for all geometries, a conservative value of $\eta = 1{,}0$ may be used.

Annex F presents an approach to determining the value of η by manual calculation. The following mapping of the variables should be applied.

a) M'_{uey}, M'_{uez} should be set to the applicable of M_{uey}, M_{uez} or M_{uay}, M_{uaz}, respectively, as described above.

b) M_{ny}, M_{nz} should be set to M_{by}, M_{bz}, respectively.

A.12.6.3.3 Interaction surface approach

In the interaction surface approach, the assessor develops a plastic strength interaction surface in terms of the axial strength and biaxial moment strengths. The interaction surface can be based on Dyer (1992)[67] and can be used for the strength checks. The approach is based on axial force applied at the "centre of squash", which is defined as the location at which the axial force produces no moment on the fully plastic section.

IMPORTANT — The assessor should be aware that the sign of the moment is crucially important for sections without material or geometric symmetry. The sign convention should, therefore, be observed with care.

NOTE A common case where errors in sign can be introduced is when taking the results of a computer analysis and applying them to a series of parametric formulae that can have a different axis convention.

A measure of the interaction ratio can, then, be obtained as the ratio between the vector length from the functional origin to the member forces, and the vector length from the functional origin to the nearest point on the surface. The functional origin is the force point associated with the functional actions in the absence of environmental actions.

Annex F provides, by way of example, conservative interaction formulae and curves for generic families of chord cross-sections based on plastic strengths P_y, M_{py}, and M_{pz}. The resistance factors should be introduced by the assessor. This is achieved by the definitions as given in Formulae (A.12.6-41) to (A.12.6-43):

$$P_y = P_{pls}/\gamma_{R,Pa} \quad \text{strength check (for all members)} \tag{A.12.6-41}$$

$$\text{or } P_y = P_p/\gamma_{R,Pa} \quad \text{beam-column check (for members subject to axial compression)}$$

$$M_{py} = M_{by}/\gamma_{R,Pb} \tag{A.12.6-42}$$

$$M_{pz} = M_{bz}/\gamma_{R,Pb} \tag{A.12.6-43}$$

where

M_{by} is the representative bending moment strength, as defined in A.12.6.3.2;

M_{bz} is the representative bending moment strength, as defined in A.12.6.3.2;

P_p is the representative axial strength of a non-circular prismatic member, as defined in A.12.6.3.2;

P_{pls} is the representative local axial strength of a non-circular prismatic member, as defined in A.12.6.3.2;

$\gamma_{R,Pb}$ is the partial resistance factor for bending strength, $\gamma_{R,Pb} = 1{,}1$;

$\gamma_{R,Pa}$ is the partial resistance factor for axial strength where

$\gamma_{R,Pa} = \gamma_{R,Pt}$ for axial tensile strength, $\gamma_{R,Pt} = 1{,}05$;

$\gamma_{R,Pa} = \gamma_{R,Pc}$ for axial compressive strength, $\gamma_{R,Pc} = 1{,}1$;

$\gamma_{R,Pa} = \gamma_{R,Pcl}$ for local strength, $\gamma_{R,Pcl} = 1{,}1$.

For the strength check, the applied member forces (P, M_y, M_z in Annex F) should be P_u, M_{uey}, M_{uez} as defined in A.12.6.3.2.

For the beam-column check, the applied member forces (P, M_y, M_z in Annex F) should be P_u, M_{uay}, M_{uaz} as defined in A.12.6.3.2.

A.12.6.3.4 Beam shear

Non-circular prismatic members subjected to beam shear forces due to factored actions should satisfy Formulae (A.12.6-44) and (A.12.6-45):

$$V_y \leq P_{vy}/\gamma_{R,Pv} \tag{A.12.6-44}$$

$$V_z \leq P_{vz}/\gamma_{R,Pv} \tag{A.12.6-45}$$

where

V_y, V_z is the beam shear due to factored actions in the local y- and z-direction, respectively;

P_{vy}, P_{vz} is the representative shear strength in the local y- and z-directions, respectively, as given in Formula (A.12.6-46):

$$P_{vy}, P_{vz} = A_v\, F_{ymin}/\sqrt{3} \tag{A.12.6-46}$$

A_v is the effective shear area in the direction being considered; see Table A.12.6-2;

$\gamma_{R,Pv}$ is the partial resistance factor for torsional and beam shear strengths, $\gamma_{R,Pv} = 1{,}1$.

Table A.12.6-2 — Effective shear area for various chord component cross-sections

Section	Effective shear area, A_v
Rolled I, H and channel sections, load parallel to web	$t\,D_s$
Welded I sections, load parallel to web	$t\,d$
Rectangular hollow sections, load parallel to webs	$A\,D_s/(D_s + B_s)$
Welded box sections, load parallel to web	$2\,t\,d$
Rolled Tee-sections, load parallel to web	$t\,D_s$
Welded Tee-sections, load parallel to web	$t\,(D_s - T)$
Circular hollow sections with diameter/wall thickness is less than or equal to 60	0,6 A
Circular hollow sections with diameter/wall thickness greater than 60	Use rational analysis
Solid bars and plates	0,9 A
Closed sections with inclined plates	$0{,}9\,\Sigma[\cos(\theta_i)\,A_{oi}]$

T is the flange thickness of a welded T-section.

t is the web thickness.

D_s is the overall depth of cross-section.

d is the web depth; for rolled sections measured with respect to root radii, for welded sections measured between inside faces of flanges.

B_s is the overall breadth of cross-section.

A is the area of cross-section.

A_{oi} is the area of rectilinear component *i*.

θ_i is the angle between *the shear force* direction being considered and the larger dimension of the cross-section of component *i*.

The effective shear area of closed-section triangular chords [see Figure A.12.1-1 b)] in the local y- and z-directions, respectively, can be calculated using elastic theory as given in Formula (A.12.6-46):

$$A_{vy}, A_{vz} = (I_v\,t_v)/Q \qquad \text{(A.12.6-46)}$$

where

I_{vy}, I_{vz} is the second moment of area of the cross section, excluding rack tooth (see A.12.3.1 and Figure A.12.1-1), about the y- and z- neutral axes, respectively;

t_{vy}, t_{vz} is the collective width of components through the cross section at the location to be assessed, perpendicular to the y- and z- axes, respectively;

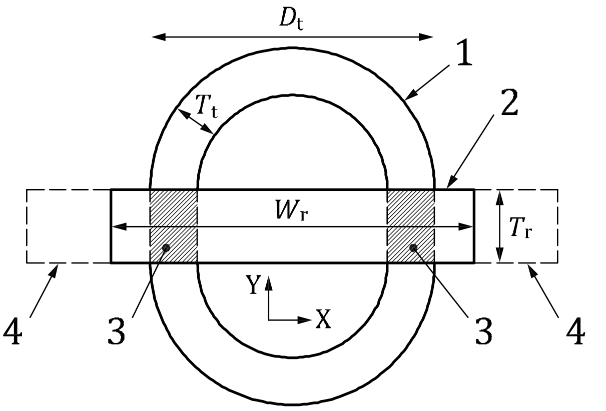
Q_y, Q_z is the first moment of the area between the cross sectional location to be assessed and the outer fibre, about the neutral axis

It can be necessary to calculate effective shear areas at various locations on the cross section in order to determine the maximum shear stress for each axis.

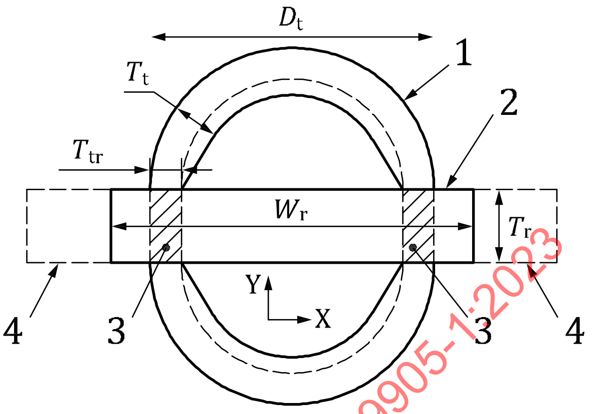
The effective shear area of a chord containing split tubular components can be calculated following the examples given in Figure A.12.6-2.

Alternatively, other rational methods can be used.

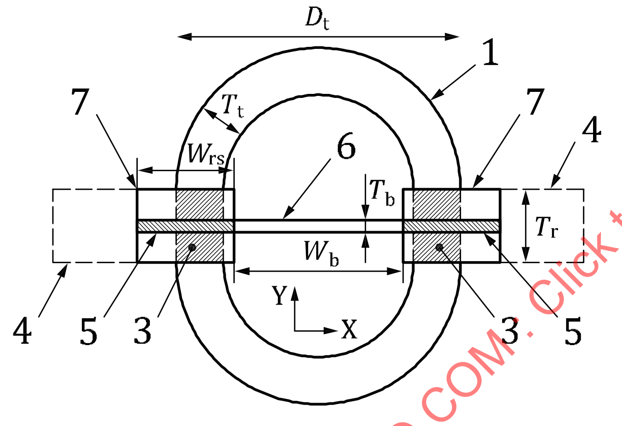
Additional information on calculating the shear stresses in a sample of split tubular and triangular cross sections can be found in ISO/TR 19905-2.



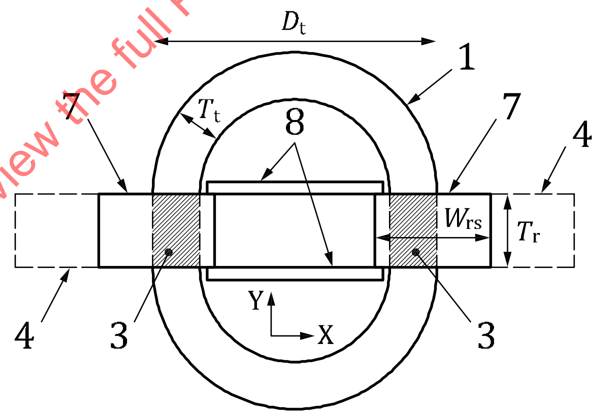
a) Split tubular with solid rack, full penetration weld of tubular to rack



b) Split tubular with solid rack, full penetration weld of tubular to rack, but tubular tapered to reduce wall thickness where connected to the rack



c) Split tubular with rack split into two parts connected by backing plate (6) with full penetration welds to back side of each rack component



d) Split tubular with rack split into two parts connected by two lapped plates (8) with fillet welds to sides of each rack component

Key

1 Structural split tubular part of chord with diameter D_t and wall thickness T_t. The area of cross section, A_t, is defined below

2 Chord rack of width W_r and thickness T_r

3 Two artificial areas that have maximum dimensions of the thickness of the rack, T_r and the width of the welded contact area between the chord tube and the chord rack, T_{tr}, with a maximum value of the chord tube wall thickness ($T_{tr} \leq T_t$), as shown in diagram (b)

4 Rack tooth, not part of structural area

5 Two artificial areas through the chord rack that have the thickness of the backing plate (item 6), T_b, and the width of the structural part of each of the two rack sections, W_{rs}

6 Backing plate of width W_b and thickness T_b that has continuous full penetration welds to the backside of each rack chord rack section

7 Two rack sections, one on each side of the chord with dimensions W_{rs} width and T_r thickness

8 Two lap plates connecting the two sections of chord rack

X Axis parallel to the chord rack

Y Axis perpendicular to the chord rack

A_t Area of cross section of the split tubular components of the member to be used in determining A_{vx} and A_{vy}. In diagrams (a), (c), and (d), A_t is the total area of cross section of both halves of the split tubular components having diameter D_t and wall thickness T_t. In diagram (b) A_t is the effective area of cross section of both halves of the split tubular components with diameter D_t but with a wall thickness T_{tr}

A_{vx} Shear area in the *X* direction. The value of A_{vx} can be determined from rational analysis. In lieu of such analysis, the following values may be used:

Diagram (a)	$A_{vx} = 0{,}9\ D_t\ T_r + 0{,}6\ A_t$
Diagram (b)	$A_{vx} = 0{,}9\ D_t\ T_r + 0{,}6\ A_t$
Diagram (c)	$A_{vx} = 0{,}6\ A_t$
Diagram (d)	$A_{vx} = 0{,}6\ A_t$

A_{vy} Shear area in the *Y* direction. The value of A_{vy} can be determined from rational analysis. In lieu of such analysis, the following values may be used:

Diagram (a, b)	$A_{vy} = 0{,}6\ A_t + 0{,}9\ T_r\ W_r$	
Diagram (c, d)	$A_{vy} = 0{,}6\ A_t + 2(0{,}9\ T_r I_r)$	In diagrams (c), & (d) $T_{tr} = T_t$

Figure A.12.6-2 — Examples of shear areas of leg chords containing split tubular components

A.12.6.3.5 Torsional shear

Closed-section non-circular prismatic members subjected to torsional shear forces due to factored actions should satisfy Formula (A.12.6-47):

$$T_u \leq T_{vp}/\gamma_{R,Pv} \tag{A.12.6-47}$$

where

T_u is the torsional moment due to factored actions;

T_{vp} is the representative torsional strength of the non-circular prismatic member as given in Formula (A.12.6-48):

$$T_{vp} = I_{pp}\ F_{ymin}/(r_t\ \sqrt{3}) \tag{A.12.6-48}$$

I_{pp} is the polar moment of inertia of the non-circular prismatic member;

r_t is the maximum distance from centroid to an extreme fibre;

$\gamma_{R,Pv}$ is the partial resistance factor for torsional and beam shear strengths, $\gamma_{R,Pv} = 1{,}1$.

Open-section non-circular prismatic members subjected to torsional shear forces should be checked as appropriate.

A.12.7 Assessment of joints

Joints should be assessed when the site conditions (metocean combinations, eccentric spudcan loading, etc.) fall outside the limits that are normally assessed by the RCS.

The designer can make joint strengths available to the assessor. When the supplied axial joint strength is less than the member strength, the supplied joint strength should be used in lieu of the member axial strength in member strength checks.

If it is considered necessary to evaluate joint strength, the resistance of tubular joints can be assessed in accordance with ISO 19902:2020, 14 and A.14 (Strength of tubular joints), and that of non-tubular joints by rational analysis. The internal forces (action effects) due to factored actions should be determined in accordance with 8.8, rather than using ISO 19902 and ISO 19901-3.

NOTE The intent of the joint check is to ensure that the joint is strong enough to resist the internal forces due to factored actions. The joint strength is not required to meet or exceed the full member strength. Guidance on non-tubular joint strength can be found in other provisions of ISO 19902 and ISO 19901-3.

A.13 Guidance on acceptance checks

No guidance is offered.

Annex B
(normative)

Summary of partial action and partial resistance factors

Table B-1 — Summary of partial action factors

Symbol	Description	Factor	Subclause(s)
$\gamma_{f,D}$	partial action factor applied to the inertial actions D_e due to dynamic response, in combination with $\gamma_{f,E}$	1,0	8.8.1.1 to 8.8.1.4
$\gamma_{f,G}$	partial action factor applied to the fixed actions G_F	1,0	8.8.1
$\gamma_{f,V}$	partial action factor applied to the actions due to variable load G_v	1,0	8.8.1
$\gamma_{f,E}$	partial action factor applied to the metocean or earthquake actions		8.8.1.1 to 8.8.1.4
	when applied to deterministic ULS storm action E_e (used with 50 year independent extreme values)	1,15	8.8.1.2
	when applied to the deterministic ULS storm action E_e (used with 100 year joint probability metocean data)	1,25	8.8.1.2
	when applied to the stochastic ULS storm actions E_e using factored metocean parameters determined in accordance with A.10.5.3.2[a]	1,0	8.8.1.3
	when applied to the inertial action induced by the ELE ground motions in earthquake analysis	0,9	8.8.1.4.1
	when applied to the inertial action induced by the ALE ground motions in earthquake analysis	1,0	8.8.1.4.2

NOTE The reference subclauses provide the methods of application and the factors are specifically tied to the calculation methodologies given in each reference subclause.

[a] The metocean partial factors used in the quasi-static stochastic analysis are determined through an iterative procedure. The procedure involves factoring the metocean parameters (wave height, current velocity and wind) until the partial-factored quasi-static stochastic force matches the action-factored quasi-static deterministic force. The start point for the iteration can be taken as $(\gamma_{f,E})^{0,5}$.

Table B-2 — Summary of partial resistance factors

Symbol	Partial resistance and material factor description	Factor	Subclause where the factor can be used
γ_{m}	partial material factor for calculated foundation capacities	1,25	E.4.6
$\gamma_{R,PRE}$	partial resistance factor for preload	1,1	A.9.3.6.2
$\gamma_{R,Hfc}$	partial resistance factor for horizontal foundation sliding capacity for effective stress (sand/drained)	1,25	A.9.3.6.2
	partial resistance factor for horizontal foundation sliding capacity for total stress (clay/undrained)	1,25	A.9.3.6.2
	partial resistance factor for horizontal foundation sliding capacity when considering material factored representative soil strength	1,0	E.4.8
$\gamma_{R,VH}$	partial resistance factor for vertical-horizontal foundation bearing capacity	1,1	A.9.3.6.4
	partial resistance factor for vertical-horizontal foundation bearing capacity when considering material factored representative soil strength	1,0	E.4.7
$\gamma_{R,Tb}$	partial resistance factor for bending strength of a tubular member[a]	1,05	A.12.5
$\gamma_{R,Tc}$	partial resistance factor for axial compressive strength of a tubular member[a]	1,15	A.12.5
$\gamma_{R,Tt}$	partial resistance factor for axial tensile strength of a tubular member[a]	1,05	A.12.5
$\gamma_{R,Tv}$	partial resistance factor for torsional and beam shear strengths of a tubular member[a]	1,05	A.12.5
$\gamma_{R,Pb}$	partial resistance factor for bending strength prismatic of a non-circular prismatic member[a]	1,1	A.12.6
$\gamma_{R,Pc}$	partial resistance factor for axial compressive strength of a non-circular prismatic member[a]	1,1	A.12.6
$\gamma_{R,Pcl}$	partial resistance factor for local axial compressive strength of a non-circular prismatic member[a]	1,1	A.12.6
$\gamma_{R,Pt}$	partial resistance factor for axial tensile strength of a non-circular prismatic member[a]	1,05	A.12.6
$\gamma_{R,Pv}$	partial resistance factor for torsional and beam shear strengths of a non-circular prismatic member	1,1	A.12.6
$\gamma_{R,S}$	partial resistance factor for spudcan strength	1,15	13.4
$\gamma_{R,H}$	partial resistance factor for holding system	1,15	13.5
$\gamma_{R,OTM}$	partial resistance factor for stabilizing moment	1,05	13.8

NOTE The requirements for partial factors are given in the normative clauses e.g. 9.3.6, 12.5, 12.6, etc. however examples of the specific application of these factors are given in Annexes A and E in the subclauses shown above. The reference subclauses provide the methods of application and the factors are specifically tied to the calculation methodologies given in each reference subclause.

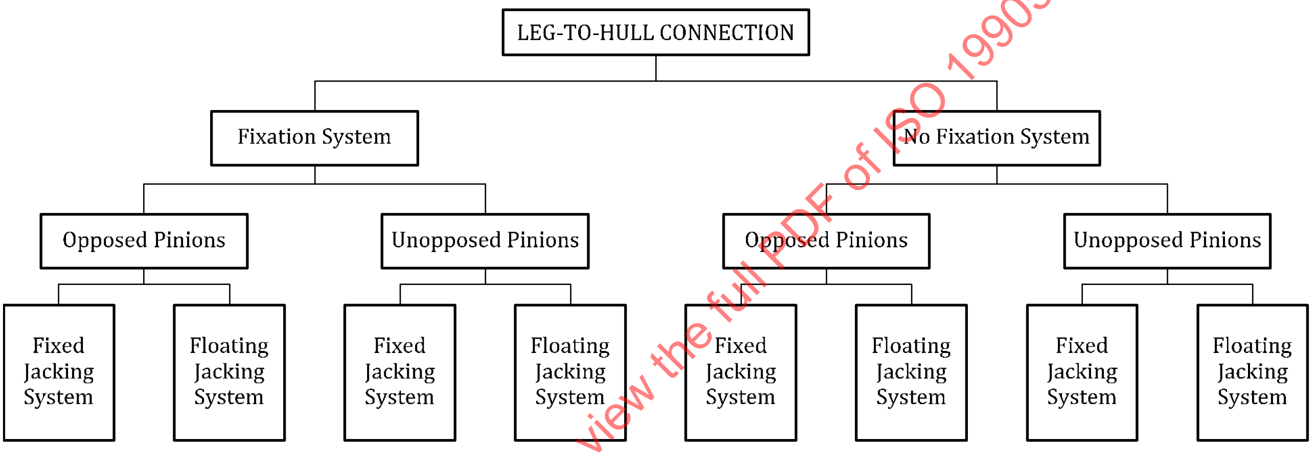
[a] The structural resistance factors for tubular members given here and referenced in Clause 12 are based on an independent interpretation of the theoretical values derived from the data used in the calibration of API RP 2A LRFD 1st edition to API RP 2A 15th edition and the data used in the development of the ISO 19902 tubular members strength formulations. The values for non-tubular prismatic members were based on AISC (2005)[13] which changed its equivalent resistance factor from 1,18 to 1,1 between the 1986 and 2005 editions because a reassessment of the applicable data resulted in an effective reduction in the coefficient of variation.

Annex C
(informative)

Additional information on structural modelling and response analysis

C.1 Guidance on 8.5 — Modelling the leg-to-hull connections

The potential leg-to-hull connection component arrangements are shown in Figure C.1-1, which also gives examples of jack-ups designs in each category.



Examples of jack-ups in each category

FandG - L780 II - JU 2000 - Alpha 350 - Super M2 - Universal M class GustoMSC - CJ 46 - CJ 50 (new) Hitachi - Giant class Keppel FELS - KFELS Mod V - KFELS Mod VI - A Class - B Class - N Class LeTourneau - Super GORILLA - Super GORILLA XL - Jaguar 250-C	GustoMSC - CJ36 - CJ46 - CJ50 (old) - CJ54 - CJ62 - CJ70	NONE	NONE	Baker Marine - Pacific 375 LeTourneau - Workhorse - Tarzan Modec - 300C - 400	CFEM - 2005 - 2600 Levingston - 111C	Baker Marine - Freedom class - 350 - 300 - 250 - 200 - 150 GustoMSC - Gusto designs LeTourneau - 53 - 84 - 82-SD-C - 116C - Super 116 - Super 116E - Super 300 - Gorilla	NONE

Figure C.1-1 — Sample leg-to-hull connection component combinations

C.2 Guidance on A.10.5.3.4 — Methods for determining the MPME

C.2.1 Guidance on the first method of Table A.10.5-1 — Fitting Weibull distributions to the results of a number of time domain simulations to determine responses at the required probability level and average the results

This procedure, outlined in Steps 1 to 7 below, requires several suitable length time domain simulations for each response of interest. The input sea state simulation should be checked for Gaussianity. Guidance is given in Table A.7.3-3. For each simulation record, the procedure for computing the MPME comprises the following steps. The final MPME value is taken as the average over all of the simulations conducted.

— **Step 1**:

The response time history, *R(t),* is first analysed to calculate the mean, μ_R, as given in Formula (C.2.1-1):

$$\mu_R = \frac{\sum_{i=1}^{n} R(t_i)}{n} \tag{C.2.1-1}$$

where

$R(t_i)$ is the time history of the response of interest;

t_i is the time point i;

n is the number of useable time points in simulation (discounting the run-in).

— **Step 2**:

The individual response maxima in the simulations are next extracted in accordance with the following criteria:

A maximum occurs at t_i if Formula (C.2.1-2) apply:

$$R(t_{i-1}) < R(t_i) \text{ and } R(t_{i+1}) \le R(t_i) \tag{C.2.1-2}$$

Suppose N_{max} maxima are found in the extraction.

— **Step 3**:

From the N_{max} response maxima, the mean of the signal, μ_R, is subtracted and the resulting maxima M_k, where k varies from 1 to N_{max}, are ranked into 20 blocks having mid-points in ascending order. The blocks all have the same width; the upper bound of block 20 is taken as 1,01 times the largest value, and the lower bound of the first block is set to zero. Any maxima with a value less than zero are discarded. The blocks are numbered in ascending order from $q = 1$ to 20, and are defined by their midpoint value M^*_q and the probability of non-exceedance of that value F_q. A distribution of the observed maxima is then found, using for each block the Gumbel plotting position in order to obtain the best possible description of the distribution for large values of M. If the number of maxima in each block, q, is n_q, the cumulative probability F_q to plot against the mid-point for block q is then as given in Formula (C.2.1-3):

$$F_q = \left[\left(1 + \sum_{j=0}^{q-1} n_j \right) \left(\sum_{j=0}^{q} n_j \right) \right]^{0,5} \Big/ \left(N_{\max} + 1 \right) \tag{C.2.1-3}$$

where n_0 is equal to the number of negative maxima peaks (the number of points not fitting into the 20 blocks).

— **Step 4 a)**:

A Weibull distribution is fitted [see Steps 4 b) to 4 d)] through the cumulative distribution of the blocks of observed maxima as defined under Step 3 [this is done in accordance with Steps 4 b) to 4 d)]. The 3-parameter Weibull cumulative distribution function is defined as given in Formula (C.2.1-4):

$$F(M^*; \alpha, \beta, \gamma) = 1 - \exp\left[-\left(\frac{M^* - \gamma}{\alpha} \right)^{\beta} \right] \tag{C.2.1-4}$$

where

$F(M^*; \alpha, \beta, \gamma)$ is the probability of non-exceedance of the value M^* where

α is the scale parameter,

β is the slope parameter,

γ is the threshold parameter.

$\alpha, \beta, (M^* - \gamma) > 0,0$

— **Step 4 b)**:

Only data blocks with a probability of non-exceedance greater than a threshold value of 0,2 are used to fit the Weibull distribution, i.e. only the blocks for which Formula (C.2.1-5) applies:

$$F_q > 0,2 \tag{C.2.1-5}$$

Notice that M_q^* are in ascending order.

— **Step 4 c)**:

For each of these blocks, q, the deviations, δ_q, of the observed probability from the corresponding probability of the Weibull cumulative distribution function, F, (transformed to Weibull scales) are calculated as given in Formula (C.2.1-6):

$$\delta_q = \ln\{-\ln[1 - F(M_q^*; \alpha, \beta, \gamma)]\} - \beta[\ln(M_q^* - \gamma) - \ln(\alpha)] \tag{C.2.1-6}$$

— **Step 4 d)**:

The parameters α, β, γ are now estimated by a non-linear least squares technique such that the summation as given in Formula (C.2.1-7) is minimized:

$$\sum_{q=x}^{20} \delta_q^2$$

where x is the value of q for which $F_q > 0{,}2$. (C.2.1-7)

The procedure may be based on a Levenberg-Marquardt algorithm, using the parameters of a 2-parameter Weibull distribution (found by the maximum likelihood method) as initial estimates.

— **Step 5**:

The MPM value, M_{MPM}, is found as the value of M for which Formula (C.2.1-8) applies:

$$F(M^*;\alpha,\beta,\gamma) = 1 - \frac{1}{\left\{N_{\max}\dfrac{T_{3\mathrm{h}}}{T_{\mathrm{sim}}}\right\}} \quad \text{(C.2.1-8)}$$

where

$T_{3\mathrm{h}}$ is 3 hours;

T_{sim} is the simulation duration.

— **Step 6**:

The corresponding MPME value, R_{MPME} is then found as given in Formula (C.2.1-9):

$$R_{\mathrm{MPME}} = \mu_{\mathrm{R}} + M_{\mathrm{MPM}} \quad \text{(C.2.1-9)}$$

where

μ_{R} is the mean value of $R(t)$ established in Step 1;

M_{MPM} is the MPME value (excluding the mean) established in Step 5.

— **Step 7**:

The procedure is repeated for each required response parameter.

C.2.2 Guidance on the second method of Table A.10.5-1: Fitting Gumbel distribution to histogram of absolute maximum responses from a number of time domain simulations to determine responses at required probability level

The basic assumption of this method is that the absolute maximum values in three-hour simulations follow a Gumbel distribution as given in Formula (C.2.2-1):

$$F_{3\mathrm{h}}(x) = \exp\left[-\exp\left(-\frac{x-\psi}{\kappa}\right)\right] \quad \text{(C.2.2-1)}$$

where

$F_{3\mathrm{h}}(x)$ is the probability that the three-hour maximum does not exceed value x;

ψ is the location parameter;

κ is the scale parameter.

The following steps are followed for each required response parameter:

— **Step 1**:

Extract absolute maximum (and minimum) value for each of at least ten three-hour response simulations.

— **Step 2**:

Fit a Gumbel distribution through these 10 or more maxima/minima. This can be done using the maximum likelihood method, yielding ψ and κ. Alternatively, Lu et al. (2001)[129] have shown that the method of moments closed form solution produces results consistent with the maximum likelihood method for values of ψ, although they showed significant variation in the values of κ. However, when calculating the MPME, with a 63 % probability of exceedance, the effects of κ approach zero, as shown in Step 3 below, and the only remaining influence is on the calculated value of ψ, as given in Formula (C.2.2-3). Therefore, the method of moments closed form solution normally can be used to calculate ψ and κ: as given in Formulae (C.2.2-2) and (C.2.2-3):

$$\kappa = (\sigma\sqrt{6})/\pi \qquad \text{(C.2.2-2)}$$

$$\psi = \mu - 0{,}577\kappa \qquad \text{(C.2.2-3)}$$

where

μ is the mean of the maxima/minima;

σ is the standard deviation of the maxima/minima.

— **Step 3**:

The value R_{MPME} is found as given in Formulae (C.2.2-4) and (C.2.2-5):

$$R_{MPME} = \psi - \kappa \ \text{In}\left\{-\ln\left[F_{3h}\left(R_{MPME}\right)\right]\right\} \qquad \text{(C.2.2-4)}$$

where

$$F_{3h}\left(R_{MPME}\right) = 0{,}37 \qquad \text{(C.2.2-5)}$$

The 0,37 lower quantile is used because the extreme of recurrence of once in three hours has a probability of exceedance of 0,63 (= 1 – 0,37). In this case, it can be seen that

$$R_{MPME} = \psi$$

— **Step 4**:

The procedure of Steps 2 and 3 can similarly be applied for minima although, because of the potential error in κ, and because the standard deviation of the minima can be large by comparison to the mean, the method of moments should not be used for calculating κ and ψ.

C.2.3 Guidance on the third method of Table A.10.5-1 — Application of Winterstein's Hermite polynomial method to the results of time domain simulation(s)

For Gaussian processes, analytical results exist for the determination of the MPM values (e.g. MPM wave height is 1,86 times the significant wave height). For general non-linear, non-Gaussian, finite band-width processes, approximate methods are used to generate the probability density function of the process. The method proposed by Winterstein (1988)[203] fits a Hermite polynomial of Gaussian processes to transform the non-linear, non-Gaussian process into a mathematically tractable probability density function. This has been further refined by Jensen (1991)[112] for processes with large kurtosis.

This procedure requires a suitable length time domain simulation for each quantity of interest. The input sea state simulation should be checked for Gaussianity. Guidance is given in Table A.7.3-3. The calculation procedure to determine the maximum of a time series, $R(t)$, with a simulation duration T_{sim} for a three-hour exposure, T_{3h}, is as follows.

— **Step 1**:

Calculate the mean, μ, the standard deviation, σ, and the following quantities of the time series for the parameter under consideration as given in Formulae (C.2.3-1) and (C.2.3-2):

$$\alpha_3 = (1/n)\Sigma[(R-\mu)^3]/\sigma^3 \quad \text{(C.2.3-1)}$$

$$\alpha_4 = (1/n)\Sigma[(R-\mu)^4]/\sigma^4 \quad \text{(C.2.3-2)}$$

where

α_3 is the skewness;

α_4 is the kurtosis.

When the kurtosis is less than 3,0, the approach given here is not valid and the alternative given in Winterstein (1988)[203] should be used.

— **Step 2**:

Construct a standardized response process, $z = (R-\mu)/\sigma$. Using this standardized process, calculate the number of zero-upcrossings, N. In lieu of an actual cycle count from the simulated time series, $N = 1\,000$ may be assumed for a three-hour simulation.

— **Step 3**:

Compute the following quantities from the characteristics of the time series for the response of interest as given in Formulae (C.2.3-3) to (C.2.3-5):

$$h_3 = \alpha_3 \Big/ \left\{4 + 2\sqrt{\left[1 + 1{,}5\left(\alpha_4 - 3\right)\right]}\right\} \quad \text{(C.2.3-3)}$$

$$h_4 = \left\{\sqrt{\left[1 + 1{,}5(\alpha_4 - 3)\right]} - 1\right\} \Big/ 18 \quad \text{(C.2.3-4)}$$

$$K = \left(1 + 2h_3^{\,2} + 6h_4^{\,2}\right)^{-1/2} \quad \text{(C.2.3-5)}$$

It is necessary to seek a more accurate result by determining a solution for C_1, C_2 and C_3 from Formulae (C.2.3-6) to (C.2.3-8):

$$\sigma^2 = C_1^2 + 6C_1C_3 + 2C_2^2 + 15C_3^2 \quad \text{(C.2.3-6)}$$

$$\sigma^3\alpha_3 = C_2(6C_1^2 + 8C_2^2 + 72C_1C_3 + 270C_3^2) \quad \text{(C.2.3-7)}$$

$$\sigma^4\alpha_4 = 60C_2^4 + 3C_1^4 + 10\,395C_3^4 + 60C_1^2C_2^2 + 4\,500C_2^2C_3^2 + 630C_1^2C_3^2 + \ldots$$
$$\ldots + 936C_1C_2^2C_3 + 3\,780C_1C_3^3 + 60C_1^3C_3 \quad \text{(C.2.3-8)}$$

using as initial guesses the values given in Formulae (C.2.3-9) to (C.2.3-11):

$$C_1 = \sigma K(1 - 3h_4) \quad \text{(C.2.3-9)}$$

$$C_2 = \sigma Kh_3 \quad \text{(C.2.3-10)}$$

$$C_3 = \sigma Kh_4 \quad \text{(C.2.3-11)}$$

where

σ is obtained from Step 1;

K, h_3 and h_4 are obtained from Formulae (C.2.3-3) to (C.2.3-4).

Following the solution for C_1, C_2 and C_3, the values for K, h_3 and h_4 are recomputed as given in Formulae (C.2.3-12) to (C.2.3-14):

$$K = (C_1 + 3C_3)/\sigma \quad \text{(C.2.3-12)}$$

$$h_3 = C_2/(\sigma K) \quad \text{(C.2.3-13)}$$

$$h_4 = C_3/(\sigma K) \quad \text{(C.2.3-14)}$$

— **Step 4**:

The most probable value, U, of the transformed process is computed as given in Formula (C.2.3-15):

$$U = \sqrt{2\ln\left(N \cdot \frac{T_{3h}}{T_{sim}}\right)} \quad \text{(C.2.3-15)}$$

where

U is a Gaussian process of zero mean, unit variance;

T_{3h} is three hours;

T_{sim} is the simulation duration, expressed in hours.

— **Step 5**:

The most probable maximum, transformed back to the standardized variable, z, is then as given in Formula (C.2.3-16):

$$z_{MPM} = K\,[U + h_3(U^2 - 1) + h_4(U^3 - 3U)] \qquad (C.2.3\text{-}16)$$

— **Step 6**:

Finally, the MPME in the required three hour exposure period for the response under consideration, can be computed from Formula (C.2.3-17):

$$R_{MPME} = \mu + \sigma z_{MPM} \qquad (C.2.3\text{-}17)$$

C.2.4 Guidance on the fourth method of Table A.10.5-1: Application of drag-inertia method to determine the base shear and overturning moment DAF from time domain simulation

The method, based on Shell International Petroleum, SIPM EPD/51/52 (1991)[167], may be used to determine $K_{DAF,RANDOM}$ used to compute the inertial loadset for a two-stage deterministic storm analysis (see 10.5.2). The method combines two components of the total dynamic response, namely the static and inertial parts. The inertial part is computed as the difference between the total dynamic and static responses and should not be confused with the response to inertial wave loading. The method requires a determination of the response of the jack-up for four conditions. In all four cases, the storm simulation (random seed) should be identical, but with different components of the loading and/or response simulated. The responses considered are usually total wave and current base shear and total wave and current overturning moment, for computing the base shear and overturning moment DAFs, respectively.

The four cases being simulated are full dynamic response, full static response, static response to inertia only wave loading (setting $C_D = 0$) and static response to drag only loading (setting $C_m = 0$). From these the inertial response is obtained as the full dynamic response minus the full static response. The means and standard deviations of the response are extracted from the time domain responses and the DAFs computed as illustrated in Figure C.2.4-1.

The drag-inertia method given here includes a final step to scale the DAF based on the period ratio T_n/T_p. This step is included to ensure that the DAF values are not underestimated for cases where T_n approaches T_p; see Perry and Mobbs (2011)[147]. The formula for the scaling factor is shown in Figure C.2.4-1 and is illustrated graphically in Figure C.2.4-2.

This method should not be used to compute MPME values for use in a one-stage stochastic analysis. It should be used only in a two-stage analysis when the foundation is modelled as either pinned or based on linearized stiffness in the DAF calculation.

Further details on the background to, and limitations of, this method can be found in ISO/TR 19905-2.

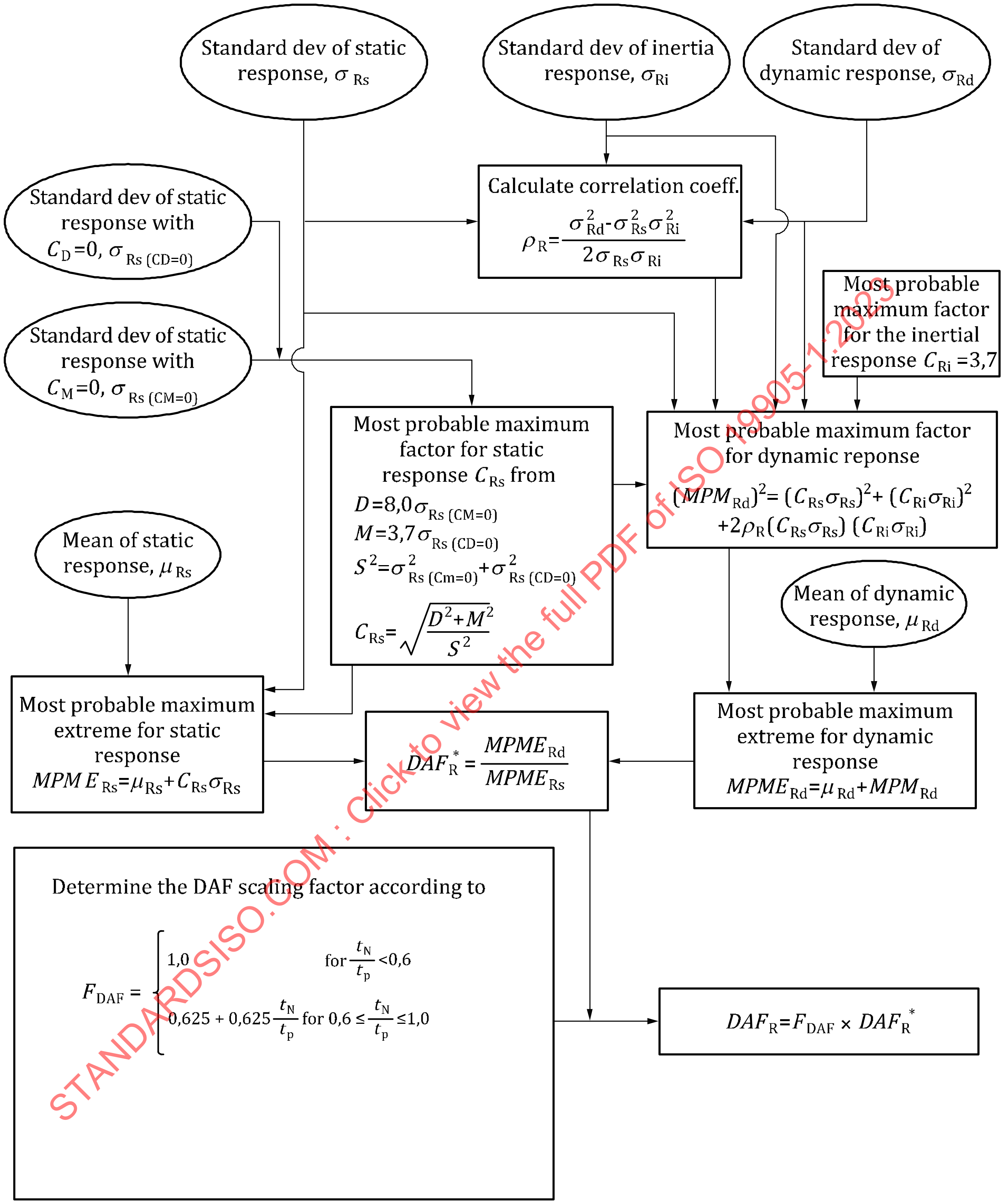


Figure C.2.4-1 — The drag-inertia method including DAF scaling factor