Petroleum and natural gas industries — Site-specific assessment of mobile offshore units —

Part 1: Jack-ups

Industries du pétrole et du gaz naturel — Évaluation spécifique au site d’unités mobiles en mer —

Partie 1: Plates-formes auto-élévatrices

ICS 75.180.10
# ISO/DIS 19905-1

## Contents

<table>
<thead>
<tr>
<th>Section</th>
<th>Title</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Scope</td>
<td>1</td>
</tr>
<tr>
<td>2</td>
<td>Normative references</td>
<td>2</td>
</tr>
<tr>
<td>3</td>
<td>Terms and definitions</td>
<td>2</td>
</tr>
<tr>
<td>4</td>
<td>Abbreviated terms and symbols</td>
<td>13</td>
</tr>
<tr>
<td>4.1</td>
<td>Abbreviated terms</td>
<td>13</td>
</tr>
<tr>
<td>4.2</td>
<td>Symbols used in Clause 8</td>
<td>14</td>
</tr>
<tr>
<td>4.3</td>
<td>Symbols used in Clause 9</td>
<td>14</td>
</tr>
<tr>
<td>4.4</td>
<td>Symbols used in Clause 10</td>
<td>15</td>
</tr>
<tr>
<td>4.5</td>
<td>Symbols used in Clause 12</td>
<td>15</td>
</tr>
<tr>
<td>4.6</td>
<td>Symbols used in Clause 13</td>
<td>15</td>
</tr>
<tr>
<td>5</td>
<td>Overall considerations</td>
<td>15</td>
</tr>
<tr>
<td>5.1</td>
<td>General</td>
<td>15</td>
</tr>
<tr>
<td>5.1.1</td>
<td>Competency</td>
<td>15</td>
</tr>
<tr>
<td>5.1.2</td>
<td>Planning</td>
<td>15</td>
</tr>
<tr>
<td>5.1.3</td>
<td>Assessment situations and associated criteria</td>
<td>16</td>
</tr>
<tr>
<td>5.1.4</td>
<td>Reporting</td>
<td>16</td>
</tr>
<tr>
<td>5.1.5</td>
<td>Regulations</td>
<td>16</td>
</tr>
<tr>
<td>5.2</td>
<td>Assessment approach</td>
<td>16</td>
</tr>
<tr>
<td>5.3</td>
<td>Selection of limit states</td>
<td>18</td>
</tr>
<tr>
<td>5.4</td>
<td>Determination of assessment situations</td>
<td>18</td>
</tr>
<tr>
<td>5.4.1</td>
<td>General</td>
<td>18</td>
</tr>
<tr>
<td>5.4.2</td>
<td>Reaction point and foundation fixity</td>
<td>19</td>
</tr>
<tr>
<td>5.4.3</td>
<td>Extreme storm event approach angle</td>
<td>19</td>
</tr>
<tr>
<td>5.4.4</td>
<td>Weights and centre of gravity</td>
<td>19</td>
</tr>
<tr>
<td>5.4.5</td>
<td>Hull elevation</td>
<td>19</td>
</tr>
<tr>
<td>5.4.6</td>
<td>Leg length reserve</td>
<td>19</td>
</tr>
<tr>
<td>5.4.7</td>
<td>Adjacent structures</td>
<td>19</td>
</tr>
<tr>
<td>5.4.8</td>
<td>Other</td>
<td>19</td>
</tr>
<tr>
<td>5.5</td>
<td>Exposure levels</td>
<td>20</td>
</tr>
<tr>
<td>5.5.1</td>
<td>General</td>
<td>20</td>
</tr>
<tr>
<td>5.5.2</td>
<td>Life-safety categories</td>
<td>20</td>
</tr>
<tr>
<td>5.5.3</td>
<td>Consequence categories</td>
<td>21</td>
</tr>
<tr>
<td>5.5.4</td>
<td>Determination of exposure level</td>
<td>22</td>
</tr>
<tr>
<td>5.6</td>
<td>Analytical tools</td>
<td>23</td>
</tr>
<tr>
<td>6</td>
<td>Data to be assembled for each site</td>
<td>24</td>
</tr>
<tr>
<td>6.1</td>
<td>Applicability</td>
<td>24</td>
</tr>
<tr>
<td>6.2</td>
<td>Rig data</td>
<td>24</td>
</tr>
<tr>
<td>6.3</td>
<td>Site data</td>
<td>24</td>
</tr>
<tr>
<td>6.4</td>
<td>Metocean data</td>
<td>24</td>
</tr>
<tr>
<td>6.5</td>
<td>Geophysical and geotechnical data</td>
<td>25</td>
</tr>
<tr>
<td>6.6</td>
<td>Earthquake data</td>
<td>26</td>
</tr>
<tr>
<td>7</td>
<td>Actions</td>
<td>26</td>
</tr>
<tr>
<td>7.1</td>
<td>Applicability</td>
<td>26</td>
</tr>
<tr>
<td>7.2</td>
<td>General</td>
<td>26</td>
</tr>
<tr>
<td>7.3</td>
<td>Metocean actions</td>
<td>27</td>
</tr>
<tr>
<td>7.3.1</td>
<td>General</td>
<td>27</td>
</tr>
</tbody>
</table>
F.1 Guidance on A.12.6.2.4: Axial compressive column buckling strength .....................................248
F.2 Guidance on A.12.6.3.2: The interaction equation approach - determination of η ....................249
F.3 Guidance on A.12.6.3.3: The interaction surface approach ..........................................................250
Annex G (informative) Contents list for typical site assessment report ...................................................260
G.1 General.................................................................................................................................................261
G.2 Jack-up and location: ........................................................................................................................261
G.3 Data check: .........................................................................................................................................261
G.4 Site assessment results summary .......................................................................................................261
G.5 Jack-up data: .......................................................................................................................................262
G.6 262
G.7 Arrangements at location .....................................................................................................................263
G.8 263
G.9 Metocean conditions ............................................................................................................................264
G.10 264
G.11 Site investigation ..............................................................................................................................264
G.12 Site hazards .......................................................................................................................................265
G.13 Soils .....................................................................................................................................................265
G.14 Analysis path/route/assumptions .......................................................................................................266
G.15 266
G.16 Spudcan fixity used in analysis .........................................................................................................266
G.17 266
G.18 Earthquake analysis ..........................................................................................................................266
G.19 Accidental situations ........................................................................................................................267
G.20 Intermediate results .........................................................................................................................267
G.21 Analysis results (utilisation checks) .................................................................................................267
Bibliography ....................................................................................................................................................268
Foreword

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

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

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

ISO 19905-1 was prepared by Technical Committee ISO/TC 67, Materials, equipment and offshore structures for petroleum, petrochemical and natural gas industries, Subcommittee SC 7, Offshore structures.

ISO 19905 is one of a series of standards for offshore structures. The full series consists of the following International Standards:

- ISO 19900, Petroleum and natural gas industries — General requirements for offshore structures
- ISO 19901-1, Petroleum and natural gas industries — Specific requirements for offshore structures — Part 1: Metocean design and operating considerations
- ISO 19901-2, Petroleum and natural gas industries — Specific requirements for offshore structures — Part 2: Seismic design procedures and criteria
- ISO 19901-3, Petroleum and natural gas industries — Specific requirements for offshore structures — Part 3: Topsides structure
- ISO 19901-4, Petroleum and natural gas industries — Specific requirements for offshore structures — Part 4: Geotechnical and foundation design considerations
- ISO 19901-5, Petroleum and natural gas industries — Specific requirements for offshore structures — Part 5: Weight control during engineering and construction
- ISO 19901-6, Petroleum and natural gas industries — Specific requirements for offshore structures — Part 6: Marine operations
- ISO 19901-7, Petroleum and natural gas industries — Specific requirements for offshore structures — Part 7: Stationkeeping systems for floating offshore structures and mobile offshore units
- ISO 19902, Petroleum and natural gas industries — Fixed steel offshore structures
- ISO 19903, Petroleum and natural gas industries — Fixed concrete offshore structures

---

1) Under preparation
2) To be published
— ISO 19904-1, Petroleum and natural gas industries — Floating offshore structures — Part 1: Monohulls, semi-submersibles and spars

— ISO 19904-2, Petroleum and natural gas industries — Floating offshore structures — Part 2: Tension leg platforms

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

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

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

3) Under preparation
Introduction

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

The series of International Standards applicable to the various types of offshore structure is intended to provide a wide latitude in the choice of structural configurations, materials and techniques without hindering innovation. Sound engineering judgement is therefore necessary in the use of these International Standards.

This Part 1 of ISO 19905 states the general principles and basic requirements for the site-specific assessment of mobile jack-ups; it is intended to be used for assessment and not for design.

Site-specific assessment is normally carried out when an existing jack-up unit is to be installed 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 standards in the ISO 19900 series, and in some cases other 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. The results of a site-specific assessment should 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 may be used provided that they have been shown to give a level of structural reliability equivalent, or superior, to that implicit in this document.

Background to and guidance on the use of this document is provided in informative Annex A, which should be read in conjunction with the main body of this document. The clause numbering in Annex A is the same as in the normative text to facilitate cross-referencing. ISO TR 19905-2 provides additional background to some clauses.

Normative Annex B summarises the partial factors. Supplementary information is presented in Annexes C - G.

To meet certain needs of industry for linking software to specific elements in this International Standard, a special numbering system has been permitted for figures, tables, equations and bibliographic references.
Petroleum and natural gas industries — Site-specific assessment of mobile offshore units — Part 1: Jack-ups

1 Scope

This part of International Standard 19905 specifies requirements and guidance for the site-specific assessment of independent leg jack-up units for use in the petroleum and natural gas industries. It addresses:

a) manned non-evacuated, manned-evacuated and unmanned jack-ups;
b) the installed phase at a specific site.

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

ISO 19901-5 does not apply specifically to mobile offshore drilling units operating in regions subject to sea ice and icebergs. When assessing a jack-up operating in such areas, the assessor should supplement the provisions of this standard with the procedures relating to ice actions and ice management contained in ISO 19906.

This document does not address design, transportation to and from site or installation and removal from site. However it is recommended that the assumptions used in the assessment be checked against the as-installed configuration.

To ensure that the design of the jack-up is sound and the structure is adequately maintained, this Part 1 of ISO 19905 is applicable only to independent leg jack-ups that either:

— hold valid classification society certification from an IACS member body 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 IACS member body.

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

NOTE 1 The IACS member, as referenced above, should meet the RCS definition given in 3.55.

NOTE 2 Future revisions of this document may be expanded to cover mat-supported jack-ups.

NOTE 3 Well conductors are 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. Annex A provides references to other publications addressing this topic.
2 Normative references

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

ISO 19900:2002, Petroleum and natural gas industries — General requirements for offshore structures

ISO 19901-1:2005, Petroleum and natural gas industries — Specific requirements for offshore structures — Part 1: Metocean design and operating conditions

ISO 19901-2:2004, Petroleum and natural gas industries — Specific requirements for offshore structures — Part 2: Seismic design procedures and criteria

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

ISO 19901-4:2003, Petroleum and natural gas industries — Specific requirements for offshore structures — Part 4: Geotechnical and foundation design criteria

ISO 19902:2007, Petroleum and natural gas industries — Fixed steel offshore structures

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

3 Terms and definitions

For the purposes of this document, the terms and definitions given in ISO 19900, ISO 19901-1, ISO 19901-2 and ISO 19901-4, and the following apply.

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

3.2 accidental situation
exceptional situation of the structure

EXAMPLES Accidental situations include impact, fire, explosion, local failure or loss of intended differential pressure (e.g. buoyancy).

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

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

NOTE An earthquake typically generates imposed accelerations.

[ISO 19900:2002]

3.4 Assessment
site-specific assessment
evaluation of the stability and structural integrity of a jack-up and, where applicable, its seabed restraint or support against the actions determined in accordance with the requirements of this document
NOTE An assessment can be limited to an evaluation of the components or members of the structure which, when removed or damaged, would cause failure of the whole structure, or a significant part of it.

3.5 assessment situation
jack-up configuration together with the environmental loading to be assessed

NOTE 1 For discussion on configuration, see 5.4.1.

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

3.6 assessor
entity performing the site-specific assessment

3.7 backfill
submerged weight of all of the soil which might be present directly above the spudcan

NOTE Backfilling might 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 preload. Both \( W_{BF,o} \) and \( W_{BF,A} \) can comprise backflow and/or infill. For discussion of the effects see A.9.3.2.2.4.

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

NOTE Backflow is part of backfill (see 3.7).

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

[ISO 19900:2002]

3.10 boundary conditions
actions and constraints on a (section of a) structural component (or a group of structural components) by other structural components or by the environment surrounding it

NOTE Boundary conditions may be used to generate reaction forces at locations of restraint.

[ISO 19902:2007]

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

NOTE Chart datum is usually an approximation to the level of the lowest astronomical tide.

[ISO 19901-1:2005]

3.12 component
see structural component
3.13 consequence category
classification system for identifying the environmental, economic and indirect personnel safety consequences of failure a jack-up

NOTE Categories for environmental and economic consequences are (see 5.3.3):

C1 high environmental and/or economic consequence,

C2 medium environmental and/or economic consequence, and

C3 low environmental and/or economic consequence.

NOTE Adapted from ISO 19902:2007, definition 3.11.

3.14 critical component
structural component, failure of which would cause failure of the whole structure, or a significant part of it

NOTE A critical component is part of the primary structure.

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

[ISO 19902:2007]

NOTE 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.

3.16 DAF\textsubscript{SDOF}
ratio of the amplitude of a dynamic action effect to the amplitude of the corresponding static action effect, each excluding their mean value

3.17 DAF\textsubscript{RANDOM}
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

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

3.19 exposure level
classification system used to define the requirements for a structure based on consideration of life-safety and of environmental and economic consequences of failure

[ISO 19902:2007]

NOTE An exposure level 1 (L1) jack-up is the most critical and exposure level 3 (L3) the least.

3.20 extreme storm event
extreme combination of wind, wave and current conditions which the structure can be subjected to during its deployment
NOTE 1 This is the metocean event used for ULS storm assessment (see Clause 6.4).

3.21 fixed load
permanent parts of the jack-up including hull, legs and spudcans, outfit, stationary and moveable-fixed equipment

NOTE 1 Moveable-fixed equipment normally includes the drilling package structure and associated permanently attached equipment.

3.22 footprint
sea floor depression which remains when a jack-up is removed from a site

3.23 foundation
soil and spudcan supporting a jack-up leg

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

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

3.26 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 When a global analysis is of a transient situation (e.g. earthquake), the inertial response is part of the equilibrium.

[ISO 19902:2007]

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

3.28 inertial loadset
a set of actions that approximates the effect of the inertial forces

NOTE An inertial loadset is used only in quasi-static analyses.

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

NOTE Infill is part of backfill (see 3.7).

3.30 intrinsic wave frequency
wave frequency of a periodic wave in a reference frame that is stationary with respect to the wave, i.e. with no current present
3.31  
**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 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.

3.32  
**jack-up owner**  
representative of the companies owning or chartering the jack-up

3.33  
**joint probability metocean data**  
combinations of wind, wave and current which produce the action effect that would be expected to occur at a site, on average, once in the return period

3.34  
**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

3.35  
**life-safety category**  
classification system for identifying the applicable level of life-safety of personnel on a jack-up:

NOTE 1 Categories for life-safety are (see 5.5.2):

S1  manned non-evacuated,

S2  manned-evacuated, and

S3  unmanned.

NOTE 2 Adapted from ISO 19902:2007, definition 3.27.

3.36  
**limit state**  
state beyond which the structure no longer fulfils the relevant assessment criteria

NOTE Adapted from ISO 19900:2002, definition 2.21.

3.37  
**long-term operation**  
operation of a jack-up on one particular site for more than the normal RCS special survey period of five years

3.38  
**lowest astronomical tide**  
LAT  
level of low tide when all harmonic components causing the tides are in phase

NOTE The harmonic components are in phase approximately once every 19 years but these conditions are approached several times each year.

[ISO 19901-1:2005]
3.39 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

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

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

3.42 mean sea level
MSL
arithmetic mean of all sea levels measured at hourly intervals over a long period, ideally 19 years

NOTE Seasonal changes in mean level can be expected in some regions and over many years the mean sea level can change.

[ISO 19901-1:2005]

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

NOTE In practice the mean zero-crossing period is often estimated from the zeroth and second moments of the wave spectrum as

\[ T_z = T_2 = \sqrt{m_0(f)/m_0(f^2)} = 2\pi \sqrt{m_0(\omega)/m_0(\omega^2)} \]

NOTE Adapted from ISO 19901-1:2005 definition 3.17

3.44 member
structural member

3.45 most probable maximum extreme
MPME
value of the maximum of a variable with the highest probability of occurring over a defined period of time (e.g. X hours)

NOTE 1 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.

NOTE 2 Adapted from ISO 19901-1:2005, definition 3.19.

3.46 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

NOTE Adapted from ISO 19902:2007, definition 3.34.
3.47 nominal stress
stress calculated in a sectional area, including 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

NOTE Overall elastic behaviour is assumed when calculating nominal stresses.

[ISO 19902:2007]

3.48 operations manual
marine operations manual
the operating manual that defines the operational characteristics and capabilities of the jack-up in accordance with the IMO MODU code

NOTE The assessor is advised to ensure that the operations manual referenced is the latest revision and that any updated weight data is provided.

3.49 operator
representative of the companies leasing the site

NOTE The operator is normally the oil company acting on behalf of co-licensees.

3.50 preloading
installation 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

NOTE Whilst 3-legged jack-ups preload by taking water ballast on board, jack-ups with 4 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.

3.51 preload reaction
preload reaction, $V_{L}$
maximum of the vertical reactions, under each spudcan, required to support the in-water weight of the jack-up during the entire preloading operation

NOTE 1 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; less the water buoyancy of the legs and spudcans (calculated from their external dimensions). Soil buoyancy and the weight of any soil backfill above the spudcan are neglected. Care needs to be taken when accounting for water contained in the spudcan (in some cases this could be included in the quoted leg weight).

NOTE 2 This is the maximum reaction that would be obtained during preloading if the jack-up were installed on a solid rock foundation.

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

3.53 quasi-static
static representation of a dynamic process

NOTE In some cases the influence of structural accelerations can be approximated by using an equivalent inertial loadset.
3.54 rack phase difference
RPD
relative position of leg chords within a leg measured along the axis of the chords

3.55 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 installations used in petroleum-related activities

NOTE Adapted from ISO 19901-7:2005, definition 3.23.

3.56 redundancy
ability of a structure to find alternative load paths following failure of one or more non-critical components, thus limiting the consequences of such failures

NOTE All structures having redundancy are statically indeterminate.

[ISO 19902:2007]

3.57 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 The term regulator can encompass more than one agency in any particular territorial waters.

NOTE 2 The regulator can appoint other agencies, such as marine classification societies, to act on its behalf, and in such cases the term regulator within this document includes such agencies.

NOTE 3 Within 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.

[ISO 19902:2007]

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

[ISO 19900:2002]

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

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

[ISO 19901-1:2005]

3.60 scatter diagram
joint probability of two or more (metocean) parameters
NOTE A scatter diagram is especially used with wave parameters in the metocean context, see ISO 19901-1 subclause 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 ($T_z$ or $T_p$).

[ISO 19901-1:2005]

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

3.62 sea state
condition of the sea during a period in which its statistics remain approximately constant

NOTE 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.

[ISO 19901-1:2005]

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

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

NOTE The significant wave height was originally defined as the mean height of the highest one-third of the 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 definition 3.31) or $4\sigma$, where $\sigma$ is the standard deviation of the time series of water surface elevation over the duration of the measurement, typically a period of approximately 30 minutes.

[ISO 19901-1:2005]

3.65 skirted spudcan
a spudcan with a peripheral skirt

3.66 slant-leg unit
jack-up with legs that can be inclined at a significant angle to the vertical

NOTE The inclination angle is typically about 5 degrees. The benefit is that the jack-up behaves more like a braced frame and less like a portal frame, with accompanying reductions in leg axial forces and moments.

3.67 sliding
horizontal movement of a spudcan

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

NOTE Also referred to as 'renewal survey' by some IACS members.
3.69
spectral density function
energy density function
spectrum
measure of the variance associated with a time-varying variable per unit frequency band and per unit directional sector

NOTE 1 Spectrum is a shorthand expression for the full and formal name of spectral density function or energy density function.

NOTE 2 The spectral density function is the variance (the mean square) of the time-varying variable concerned in each frequency band and directional sector. Therefore the spectrum is in general written with two arguments: one for the frequency variable and one for a direction variable.

NOTE 3 Within this document 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.

[ISO 19901-1:2005]

3.70
spectral peak period
period of the maximum (peak) energy density in the spectrum

NOTE In practice there is often more than one peak in a spectrum.

[ISO 19901-1:2005]

3.71
spudcan
structure at the base of a leg supported by the soil

3.72
squeezing
lateral movement of weak soil between the spudcan base and an underlying stronger layer or of weak soil between two stronger layers

3.73
stochastic analysis
analysis in which a probabilistic approach is taken to model the random nature of the variables of interest

NOTE In general, a linear(ized) stochastic analysis can be performed in the frequency domain or in the time domain whereas nonlinear stochastic analysis can only be performed by time domain simulations. This document does not support frequency domain stochastic analysis.

3.74
stress concentration factor
SCF
factor relating a nominal stress to the local stress at a detail

[ISO 19902:2007]

3.75
structural analysis
process or algorithm for determining action effects from a given set of actions

NOTE 1 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.

NOTE 2 Adapted from ISO 19902:2007 definition 3.51
3.76 **structural component**

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

**NOTE**  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 may be of a different yield strength. See also further discussion in A.12.1.1.

3.77 **structural member**

physically distinguishable part of a braced structure connecting two joints

or

leg of a non-truss leg jack-up

**NOTE**  See also further discussion in A.12.1.1.

3.78 **sudden hurricane**

**sudden cyclone**

**sudden typhoon**

sudden tropical revolving storm which forms near the site and can affect the jack-up before demanning can be completed within the time required by the emergency evacuation plan

**NOTE**  The intent is that the jack-up be assessed to L1 for the specified sudden tropical revolving storm.

3.79 **sustained wind speed**

time averaged wind speed with a defined averaging duration of one minute or longer

**NOTE**  In ISO 19901-1:2005 definition 3.33 references a duration of “ten minutes or longer”

3.80 **undrained shear strength**

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

**NOTE**  Adapted from ISO 19901-4:2003 definition 3.9.

3.81 **utilization**

**member utilization**

**foundations utilization**

(maximum 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**  The utilization is the maximum absolute value of the ratio for each limit state and assessment situation being considered.

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

**NOTE 3**  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**  For members and foundations subjected to combined forces, the internal force pattern and the resistance combine into an interaction equation. If the interaction equation governing the assessment check is, or can be, reduced to an inequality of the form \( I \leq 1,0 \) the utilization is equal to \( I \).

**NOTE 5**  Adapted from ISO 19902:2007, definition 3.57.
3.82 **variable load**
items carried by the jack-up to support its operation that are not included in the fixed load

3.83 **water depth**
vertical distance between the sea floor and still water level

**NOTE 1** 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** 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.

**NOTE 3** Adapted from ISO 19901-1:2005, definition 3.41.

3.84 **working stress**
to be defined if and when required by Dave

4 **Abbreviated terms and symbols**

4.1 **Abbreviated terms**

**ALE** abnormal level earthquake

**ALS** accidental limit state

**BOP** blow-out preventer

**BSTF** base shear transfer function

**CD** chart datum

**DAF** dynamic amplification factor

**DAF\textsubscript{SDOF}** DAF obtained excluding the mean values (typically from a single degree of freedom calculation)

**DAF\textsubscript{RANDOM}** DAF obtained including the mean values (typically from a random wave calculation)

**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

**MHWS** mean high water spring

**MLWS** mean low water spring
MPME  most probable maximum extreme
MSL   mean sea level
OCR   over-consolidation ratio
PDF   probability density function
PCPT  piezocone penetrometer test
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
SWL   still water level
TRS   tropical revolving storm
ULS   ultimate limit state
WSD   working stress design

4.2 Symbols used in Clause 8

\( D_e \) equivalent set of indirect actions representing dynamic extreme storm effects
\( E_e \) metocean action due to the extreme storm event
\( F_d \) assessment load case
\( G \) actions due to the fixed load positioned to 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
\( \gamma_{1D} \) partial action factor applied to the inertial actions due to dynamic response
\( \gamma_{1E} \) partial action factor applied to the metocean or earthquake actions
\( \gamma_{1G} \) partial action factor applied to the actions due to fixed load
\( \gamma_{1V} \) partial action factor applied to the actions due to the variable load

4.3 Symbols used in Clause 9

\( W_{BF,A} \) submerged weight of backfill that occurs after the maximum preload has been applied
4.4 **Symbols used in Clause 10**

$f_{FD}$ fatigue design factor

4.5 **Symbols used in Clause 12**

$C_m$ moment reduction factors

4.6 **Symbols used in Clause 13**

$A_E$ action effect

$F_d$ factored action

$M_{OTM}$ overturning moment due to factored actions

$O$ representative capacity

$R$ factored resistance

$R_{dOTM}$ the factored stabilizing moment

$R_{r,OTM}$ representative stabilizing moment

$U$ utilisation

$U_{S,vhm}$ the foundation bearing capacity utilization

$U_{S,hvm}$ the foundation sliding resistance utilization

$U_{S,pl}$ preload utilization

$\gamma_f$ partial action factor

$\gamma_R$ the resistance factor for each representative capacity

$\gamma_{R,OTM}$ the resistance factor on representative stabilizing moment

5 **Overall considerations**

5.1 **General**

5.1.1 **Competency**

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

5.1.2 **Planning**

Adequate planning shall be undertaken before a site-specific assessment is started. The planning shall include the determination of all assessment situations and the criteria upon which the assessment shall be based, following the general requirements specified ISO 19900 as far as relevant for jack-ups.
5.1.3 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.

5.1.4 Reporting

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

5.1.5 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 comply with relevant rules and regulations, depending upon the site and type of operations to be conducted.

5.2 Assessment approach

This subclause provides an overview of the data required, the assessment methodology, and the acceptance criteria. A flow chart is shown in Figure 5.2-1. Annex A (informative) provides additional information and guidance, including detailed calculation methodology. Annex B (normative) provides the partial factors to be used in the assessment. Annexes C to F (informative) provide supplementary information or alternative calculation methodologies. Annex G provides a recommended contents list for the assessment report. The associated Technical Report, ISO TR 19905-2 provides background to some of the recommendations given in the Annexes. Other approaches 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 need 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 according to 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 according to the more complex methods (e.g. secant, yield interaction or continuum foundation model, time-domain dynamics) given in this document.
Figure 5.2-1 — Overall flow chart for the assessment
5.3 Selection of limit states

ISO 19900 divides the limit states into four categories as described below. As described below, normally only the ULS needs to be assessed in a jack-up site-specific assessment.

a) Ultimate limit state (ULS).

The site-specific assessment shall include evaluation of the ULS for assessment situations including extreme combinations of metocean actions and the associated storm mode gravity actions. Earthquake actions shall also be considered in combination with the associated operational mode gravity actions; however evaluation is rarely required. The applicable partial action and resistance factors for the ULS and exposure level are summarized in Normative Annex B. Under the action of the ULS, the integrity of the structure should be unimpaired, but damage to non safety-critical (secondary) structure of the jack-up may be tolerated.

When the ULS metocean conditions are within the defined SLS limits for the jack-up (i.e. the metocean conditions are less severe than those defined for changing to the elevated storm configuration), this ULS situation shall be assessed with the jack-up the most critical operating configuration (increased variable load, cantilever extended and unequal leg loads). This 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.

Similarly, 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 perform the ULS assessment using the operational metocean conditions with the associated operating mode gravity actions and configuration. 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 ULS metocean conditions are within the defined SLS limits for the jack-up and do not exceed the limits for changing the jack-up to the elevated storm configuration.

b) Serviceability limit state (SLS).

The SLS is normally covered by the limits specified in the operations manual, and therefore does not need to be assessed unless the operational configuration requirements for the site are outside those limits. However, the requirements of a) above always apply.

c) Fatigue limit state (FLS).

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

d) Accidental limit states (ALS).

The ALS are generally addressed at the design stage and do not need to be evaluated in the assessment unless there are unusual risks at the site under consideration (e.g. when there is a need to perform an ALE analysis).

5.4 Determination of assessment situations

5.4.1 General

A jack-up can be used in various modes at a single location (e.g. drilling mode / workover mode / tender 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 appropriate acceptance criteria, 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 the personnel operating the jack-up and to any other relevant stakeholders. 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 may be assumed to be pinned at the reaction point. Any divergence from this assumption shall be stated.

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 vs. 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 be used 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 comply 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 prediction of penetration and to provide a contingency against settlement or scour. Recommended leg length reserve requirements are given in 13.7.

5.4.7 Adjacent structures

The interaction of the jack-up with any adjacent structures shall be considered and reported, as appropriate. Aspects requiring consideration by one or more of the stakeholders include 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 estimate of the conditions at the site. In some cases it can be found that the actual conditions are inconsistent with the assumptions made 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.
5.5 Exposure levels

5.5.1 General

Structures can be categorized by various levels of exposure to determine criteria that are appropriate for the intended service of the structure. The levels are determined by consideration of life-safety and of environmental and economic consequences of failure.

The life-safety category addresses personnel on the jack-up and the likelihood of successful evacuation before an extreme storm event occurs.

The consequence category considers the potential risk to life of personnel brought in to react to any incident, the potential risk of environmental damage and the potential risk of economic losses.

5.5.2 Life-safety categories

The category for life-safety (S1, S2 or S3) shall be determined by the jack-up owner prior to the assessment. When either S2 or S3 is selected this shall be agreed with the operator and, where applicable, the regulator. It is recognized that matching actual situations to generic life-safety category definitions requires a degree of judgement.

a) S1 Manned non-evacuated

The manned non-evacuated category refers to the situation when a jack-up (or an adjacent structure that could be affected by the failure of the jack-up) is continuously manned, and from which personnel evacuation prior to the extreme storm event is either not intended or impractical.

A jack-up shall be categorized as S1 manned non-evacuated unless the particular requirements for S2 or S3 apply throughout the expected period of operations at the assessment location.

A jack-up shall always be considered S1 for the consideration of earthquake events.

b) S2 Manned evacuated

The manned-evacuated category refers to a jack-up that is normally manned except during a forecast extreme storm event. For categorization purposes, a manned jack-up may only be categorized as a manned evacuated jack-up if:

1) reliable forecasting of a extreme storm event is technically and operationally feasible, and the weather between any such forecast and the occurrence of the extreme storm event is not likely to inhibit an evacuation; and

2) prior to an extreme storm event, evacuation is planned; and

3) sufficient time and resources exist to safely evacuate all personnel from the jack-up (and any adjacent structure that could be affected by the failure of the jack-up) with due consideration of the other demands on those resources (e.g. the evacuation of other manned platforms in the area).

c) S3 Unmanned

The unmanned category refers to a jack-up that is only manned for occasional inspection, maintenance and modification visits. For categorization purposes a jack-up shall only be categorized as unmanned if:

1) visits to the jack-up are undertaken for specific planned inspection, maintenance or modification operations on the jack-up; and
2) visits are not usually expected to last more than 24 hours during seasons when severe weather can be expected to occur; and

3) the evacuation criteria for S2 manned evacuated jack-ups are met.

A jack-up in this category may also be described as "not normally manned".

5.5.3 Consequence categories

Factors that should be considered in determining the consequence category include:

- life-safety of personnel on, or near to, the jack-up who are brought in to react to any consequence of failure, but not personnel that are part of the normal complement of the jack-up,
- damage to the environment, and
- anticipated losses to the jack-up owner, to the operator, to the industry and/or to other third parties as well as to society in general.

NOTE This classification includes risk of loss of human life for people other than the jack-up's normal complement. The primary driver for the classification is damage to the environment or to society (e.g. the situation where a community/state/country would suffer significant losses as a consequence of the interruption of production). The classification is based on the assumption that all stakeholders agree on the economic loss category to suit their tolerance of risk.

The consequence category that applies shall be determined by the jack-up owner prior to the assessment and shall be agreed by the operator and, where applicable, the regulator and operator(s) of adjacent facilities. It is recognized that matching actual situations to generic consequence category definitions requires a degree of judgement.

a) C1 High consequence category

The high consequence category refers to jack-ups where the failure of the jack-up has the potential to cause high risk to emergency response personnel and/or high consequences in terms of environmental damage and/or economic loss.

Unless the above conditions apply a jack-up shall normally be categorized as C2 or C3.

NOTE 1 Adjacent facilities (workover platform, local platforms, transport lines, etc.) are those that are sufficiently close to the jack-up location for there to be a high probability of impact if the jack-up collapses or drifts from location. They are unlikely to be "high consequence", although they could have been designed to a higher categorisation than is applicable during the specific jack-up operation being assessed. In most cases facility damage does not result in significant reduction in throughput or hydrocarbon production and the facility has the protection to meet C2 or C3 requirements. The cost of plugging and abandoning the wells can be significant and raise the consequence category.

NOTE 2 Examples of high consequences include the significant unintended release of hydrocarbons from the well(s) or from adjacent major transport lines and/or storage facilities.

NOTE 3 Where the shut-in of hydrocarbon production is not planned, or not practical, prior to the occurrence of an extreme storm event the location can be high consequence.

NOTE 4 All earthquake events are considered to be high consequence because of life-safety, see S1.

b) C2 Medium consequence category

The medium consequence category refers to jack-ups where production of hydrocarbons on both the jack-up and any adjacent facility is shut-in during the extreme storm event. All the following criteria shall apply:
1) all wells that could flow on their own in the event of structural or foundation failure shall contain fully functional means of reliably closing in the well to prevent such flow, and such means shall be manufactured and tested in accordance with applicable specifications;

The possibility of flow should be considered as a result of failure in any part of the system including the riser/conductor.

2) oil storage is limited to process inventory and “surge” tanks for pipeline transfer;

3) pipelines that could be affected by failure of the jack-up shall be protected from releasing hydrocarbons, either by virtue of inventory and pressure regime, or by check valves or seabed safety valves located at sufficient distance to be unaffected by the failure;

4) the failure of the jack-up is evaluated to cause medium or low consequences to any facility it is operating over, or adjacent to.

c) C3 Low consequence category

The low consequence category refers to jack-ups operating in:

- open water locations with no existing surface or subsea infrastructure, or
- workover mode or production mode with low production rates and where any production is shut-in during the extreme storm event.

These units may support production departing from the jack-up and low volume infield pipelines. For categorization purposes a jack-up shall only be categorized as low consequence if:

1) all wells that could flow on their own in the event of structural or foundation failure shall contain fully functional means of reliably closing in the well to prevent such flow., and such means shall be manufactured and tested in accordance with applicable specifications;

The possibility of flow should be considered as a result of failure in any part of the system including the riser/conductor.

2) oil storage is limited to process inventory;

3) pipelines that could be affected by failure of the jack-up shall be limited in their ability to release hydrocarbons, either by virtue of inventory and pressure regime, or by check valves or seabed safety valves located at sufficient distance to be unaffected by the failure;

4) the failure of the jack-up is evaluated to cause low consequences to any facility it is operating over, or adjacent to.

5.5.4 Determination of exposure level

The three categories for each of life-safety and consequence can, in principle, be combined into nine exposure levels. However, the level to be used for categorization is the more restrictive level for either life-safety or consequence. This results in three exposure levels as illustrated in Table 5.5-1.

The exposure level applicable to a jack-up shall be determined by the jack-up owner prior to the assessment and, where applicable, shall be agreed by the regulator and operator and by the regulator and operator(s) of adjacent facilities.

For extreme storm assessments, the metocean criteria and the associated partial action factors for the exposure levels are given below:
Table 5.5-1 — Determination of exposure level

<table>
<thead>
<tr>
<th>Life-safety Category</th>
<th>Consequence category</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>C1 High consequence</td>
</tr>
<tr>
<td>S1 Manned non-evacuated</td>
<td>L1</td>
</tr>
<tr>
<td>S2 Manned evacuated</td>
<td>L1</td>
</tr>
<tr>
<td>S3 Unmanned</td>
<td>L1</td>
</tr>
</tbody>
</table>

L1 : A manned or C1 jack-up shall be assessed for either the 50 year independent extremes with partial action factor = 1.15 or for the 100 year joint probability metocean data with partial action factor = 1.25.

L2 : A lower consequence manned-evacuated jack-up shall be assessed for the 50 year independent extremes or 100 year joint probability metocean data that could be reached at the site prior to evacuation being effected (e.g. 48 hour sudden hurricane in Gulf of Mexico). The assessment shall use the partial factors applicable to L1.

The L3 condition shall also be considered for the post-evacuation, unmanned, case.

L3 : The unmanned, low-consequence (survivability) criteria, to be agreed between the stakeholders which would normally include the jack-up owner, operator, regulator.

NOTE << It is expected that we will say: >>Metocean data for L2 and L3 and factors for L3 applicable for the Gulf of Mexico can be found in Regional Annex zz.

For earthquake a jack-up shall be assessed as L1 using 1000 year earthquake event.

<< Note from WG7 to the reader: This draft international standard has been prepared for site-specific analysis of the jack-up platform using best available practices from current technology. The methods presented are being subjected to a comprehensive benchmarking process to confirm that results are reasonably consistent with current best practice. Until this process is complete, the standard should not be utilized as an exclusive resource for jack-up site assessment. >>

5.6 Analytical tools

Most of the analytical procedures and calculations described in this part of ISO 19905 are commonly performed with the assistance of computer-aided engineering tools. Many of these, particularly structural analysis programs, consist of recognized commercially available software suites which, when used by experienced and well trained operators, can be considered suitable for their standard areas of application. For these software systems, the original author is expected to have performed adequate validation and verification for their standard areas of application, and to maintain evidence thereof. However, many of these software suites do not adequately address jack-up specific issues, such as time-domain dynamics, foundations, large displacement effects and code checks.

In cases where innovative analytical approaches and techniques are used with commercially available software suites or where proprietary software solutions are adopted, the assessor is expected to validate the adequacy of methodology.
6 Data to be assembled for each site

6.1 Applicability

This clause describes the data that are required to undertake an assessment. In this document the site is the general area where the jack-up is to operate; the location is the specific position/orientation within the site. The location data is normally a sub-set of the site 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 Rig data

The jack-up data required to perform an assessment includes:

- rig type;
- installed leg length;
- drawings, specifications and the latest revision of the operations manual;
- data pertaining to the strength, stiffness and operation of the leg-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 and any proposed deviations for the intended operation;
- details of any relevant modifications.

6.3 Site data

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

At platform locations, platform drawings, the required hull elevation or the required clearances with the platform, the rig heading, and other interface data shall be obtained from the platform operator.

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:

- water depth (LAT or CD):
- tide and storm surge;
- wave data
  - significant wave height and spectral peak period;
  - maximum wave height and associated period;
— abnormal wave crest elevation (see A.6.4.2.4);
— current velocity and profile;
— wind speed.

Further reference to metocean data can be found in Tables A.7.3-1 and A.7.3-2.

Omnidirectional data may be sufficient, but in particular circumstances directional data may also be required.

Other data shall be evaluated when applicable e.g.:

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

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

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

As a minimum, a manned-evacuated jack-up shall be assessed for the 50 year independent extremes or 100 year joint probability metocean data that could be reached while the jack-up is still manned, but see Table 5.5-1. For example in the TRS areas consideration may be given to the use of a 50 year return period “sudden hurricane” or “sudden storm” event.

As a minimum, an unmanned jack-up shall be assessed to an agreed survivability criteria, but see Table 5.5-1.

If the jack-up deployment is to be of limited duration, applicable seasonal data may be used (for example, the assessment return period summer extreme storm event).

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 shortcrestedness 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.

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 depends on the particular circumstances such as the type of jack-up and previous experience at the location, locations within 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 diver’s survey data.

The site shall be evaluated for the presence of geohazards as described in Table A.6.5-1.

For sites where previous operations have been performed by jack-ups of the same basic design, it may be sufficient to identify the location of, and hazards associated with, existing footprints and refer to previous site data and preloading or penetration records; however, it is recommended that the accuracy of such information should be verified.
At sites where there is any uncertainty, borings/corings and/or piezocone penetrometer tests (PCPT) data are recommended at the planned location. Alternatively, the site may be tied-in to such data at another site by means of shallow seismic data. If data are not available prior to the arrival of the jack-up, it may 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 location and during subsequent preloading.

The site shall be evaluated for potential scour problems. These are most likely to occur at sites with a firm seabed composed of non-cohesive soils and where the penetration is low.

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.

7 Actions

7.1 Applicability

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

Details regarding applicable methods and formulations 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 the present clause, and the corresponding annex, actions are presented without partial action factors. Actions shall be factored as given in 8.8 prior to the determination of the action effects.

7.2 General

The following outlines the actions to be considered in general terms:

a) Metocean actions
   1) Actions on legs and other structures subject to wave and current action, plus
   2) Actions on hull and exposed areas (e.g. legs) subject to wind action.

b) Functional actions
   1) Fixed actions, plus
   2) Actions from variable load

c) Indirect actions resulting from responses
   1) Displacement dependent effects, plus
   2) Accelerations from dynamic response.

d) 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 collinearity should normally be assumed. The directionality of collinear 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 wind, 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. 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 periods (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 \( T_i \), see A.7.3.3.1 and ISO 19901-1 Clauses 8.3 and A.8.3.

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.

7.3.4 Wind actions

Wind actions shall be computed using wind velocity, wind profile and exposed areas. Wind velocities and wind profiles presented in A.6.4.6 shall be used. 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.

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 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 location and communicate them to the jack-up owner. The intent is to ensure that these limits are included in the operating procedures for the location.
7.5 Displacement dependent effects

Indirect forces that are a consequence of the displacement of the structure, P-\(\Delta\) effects, shall be considered in the analysis. The P-\(\Delta\) effects are due to the first order sway, 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.

7.7 Earthquakes

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

7.8 Other actions

Additional leg moments due to leg inclination resulting from leg-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, 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 a 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.

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 the static and dynamic responses:

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

- Suitable representation of the leg, leg/hull connection and the leg-foundation interaction, including nonlinear effects as necessary; and

- Sufficient detail to allow for 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 forces and displacements can be obtained through use of simplified, equivalent modelling techniques.

To determine displacements and forces in the leg, leg/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.

8.2.3 Levels of FE modelling

In general, a jack-up model shall include the leg, leg/hull connection and representative hull structure. FE models can contain combinations of detailed and simplified structural modelling. Four modelling techniques are summarized below.

a) Fully detailed model of all legs and leg/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/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/hull connections with detailed or representative stiffness model of hull.

d) Detailed single leg (or leg section) and leg/hull connection model. This model is to be used in conjunction with the reactions at the spudcan or the forces and moments in the vicinity of 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 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-hull clearances and hull inclination shall be considered (see 10.5.4), but need not 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/hull connection

8.5.1 General

The leg/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

The fixation system 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/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 location shall be determined from applicable analyses.

8.6.2 Seabed reaction point

Selection of the reaction point shall be based on the estimated penetration using geotechnical information from the site 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/hull connection.

In cases where the inclusion of rotational foundation fixity is justified and is included in the structural analysis, the nonlinear 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.

When assessing the spudcans, the leg-to-can connection and the lower parts of the leg, the spudcan reactions shall be obtained from a foundation model that properly estimates the spudcan moment.

For earthquake excitation, foundation fixity tends to increase the inertial response and shall be considered. Spudcan settlement resulting from earthquake excitation shall be considered. Differential settlements can have the most serious consequences.

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 and added mass. 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 within the hull) shall be included in the mass model. Other actions due to variable load such as conductor tension and hook loads, that are not associated with masses should not be included.

The structural mass shall include:

- legs;
The non-structural mass shall include:

- hull equipment and outfitting;
- the mass of the variable load;
- sea water supply system;
- leg appurtenances;
- marine growth; and
- entrapped water in flooded members and spudcans.

Added mass shall include contributions from:

- submerged leg chords and braces;
- sea water caissons; and
- for earthquake assessments only, spudcans, see A.8.7.

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 nonlinear behaviour is properly captured.

\[
F_d = \gamma_{G}G + \gamma_{Gv}G_v + \gamma_{Ee}E_e + \gamma_{De}D_e
\]  

(8.8-1)

where the actions are defined as:

\( G \) = actions due to the fixed load positioned to represent their vertical and horizontal distribution, see 8.8.2.

\( G_v \) = 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, see 8.8.2.

\( E_e \) = metocean action due to the extreme storm event, see 8.8.4, or

\( = \) zero for earthquake assessment

\( D_e \) = equivalent set of indirect actions representing dynamic extreme storm effects, see 8.8.5, or

\( = \) zero for stochastic storm assessment according to 10.5.3, or
= inertial actions induced by the ELE and ALE ground-motion for earthquake assessment, see 8.8.8.

Where the partial action factors $\gamma$ are given in 8.8.1.2 - 8.8.1.4 below.

NOTE Reference can be made to the Table in Annex B which contains all of the applicable factors to be used in a site-specific analysis.

The actions and action effects to 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 are 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 50 year return period independent extreme metocean actions

= 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

The stochastic storm analysis described in 10.5.3 is carried out using unfactored actions, resulting in the partial action factors 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 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

The partial action factors for ELE analysis described in 10.7 are given below:

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

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

$\gamma_{1,E} = 0,9$ when applied to the ELE actions

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

The partial action factors for the ALE are:

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

$\gamma_{1,V} = 1,0$ and is applied to the actions due to the variable load
8.8.2 Functional actions due to fixed load and variable load

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, and
- buoyancy.

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; and
- 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 loading 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 the 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 the 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 can be 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 (sidesway) under environmental actions.

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 shall be included in the analysis.

8.8.8 Earthquake actions

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.

Earthquake actions shall include global accelerations due to the fundamental modes of vibration in addition to the local actions from soil movement on the spudcans and the legs, where relevant. The associated inertial actions on all significant masses shall be taken into account.

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 should always take precedence in case of conflict.

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

9.2 General

Adequate geotechnical and geophysical information shall be gathered to assess the spudcan penetration and foundation stability of the jack-up at the location. Applicable information from previous operations, other surveys or activities in the area should be used in the assessment of the location. Soil investigation shall be carried out for locations where the available data are not applicable or inadequate. See 6.5 and A.6.5 for details of the recommended geotechnical and geophysical information.

There are two objectives of this assessment. The first is to ensure that the foundation is adequate to carry static, cyclic, and transient forces without excessive settlement or movement. The assessment shall consider:

- the predicted leg penetration;
- the possibility of rapid leg penetration and/or punch through;
- likely scale of spudcan movements;
- the effects of cyclic loading;
- the consequences of specific site conditions, such as are listed in 9.4.

The second objective is to provide foundation models of increasing sophistication for use in structural response analyses.

9.3 Geotechnical analysis of independent leg foundations

9.3.1 Foundation modelling and assessment

The forces imposed on the foundation due to environmental loading 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
The leg is nonlinear and hysteretic. The nonlinearity of the foundation response can have a major effect on the dynamic 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 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:

- simple pinned foundation (pinned model);
- linear vertical, linear horizontal and secant rotational stiffness where the iterative reduction of rotational stiffness ensures compliance with the yield interaction surface (secant model);
- nonlinear vertical, horizontal and rotational stiffness model where the nonlinear behaviour ensures compliance with the yield interaction surface (yield interaction model);
- nonlinear continuum foundation model coupled to the structure (continuum model); 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 Clause 9 below.

### 9.3.2 Leg penetration during preloading

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 there is insufficient preload capacity to meet the assessment reactions, a lower preload may be acceptable when justified (e.g. by the level 3 displacement check in 9.3.6).

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 equations adopted are based on the assumption that penetration in sand is a drained process, and penetration in clay is an undrained 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 and A.9.4.9.

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 these data during rig move operations 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 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 foundation 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 expanding its yield surface and increasing its bearing 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 location. In-elastic response occurs when the combination of vertical, horizontal and moment loading approaches the yield surface; this is only likely for a few, if any, extreme events. Degradation can take the form of a softened foundation, additional penetration or both.

The yield surface can be described by the equations 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 spudcan shall be included in the spudcan vertical reaction to be assessed against the capacity envelopes.

9.3.4 Foundation stiffness

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 (A.9.3.4.3) or sand (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.9.3.4.2.3 shall 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 nonlinear continuum foundation models, as discussed in A.9.3.4.2.4 and A.9.3.4.2.5, 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 may 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 compliance with the yield interaction model.

The envelopes shall be developed using the applicable subclause of A.9.3.5. The weight of all soil backfill directly above the spudcan shall be included in the spudcan vertical reaction when evaluating the capacity envelopes.

9.3.6 Acceptance checks

The overall jack-up foundation stability shall be assessed using levels 1, 2 or 3, as listed below (in order of increasing complexity and reducing conservatism), see Figure A.9.4-13. 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.

Level 1 Preload and sliding check with reactions from a pinned spudcan analysis; steps 1a and 1b shall both be completed for a level 1 check.
Step 1a The foundation capacity check is based on the preloading capability (A.9.3.6.2), and

Step 1b Sliding of the windward leg is also checked (A.9.3.6.3).

Level 2 Capacity checks. One of the following three steps shall be completed for a level 2 check:

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

Step 2b Bearing capacity and sliding resistance check (A.9.3.6.5), including rotational, vertical and horizontal foundation stiffness with rotational stiffness reduction; or

Step 2c Bearing capacity check (A.9.3.6.5), including rotational, vertical and horizontal foundation stiffness with reduction of vertical, horizontal and rotational stiffnesses. A level 3 displacement check shall be performed.

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 leg-penetration curve based on the results of a level 2 check (not permitted if level 2 sliding check is failed); or

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

Under pinned conditions, 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 1, the preload check of the leeward leg is based on the assumption that the 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 \( W_{BF,A} \) is uncertain; for this reason, it should conservatively be included on the leeward leg but not the windward leg. The sliding check of the windward leg shall also be performed to ensure that the sliding resistance is adequate under minimum vertical reaction conditions. (See 7.2 on selection of load cases).

In step 2a, the combined vertical and horizontal leg reactions shall be checked against the factored bearing capacity of the leeward leg and sliding capacity of the windward leg. 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. The moments are ignored in step 2a analyses as the spudcans are considered to be pinned.

For step 2b, the combined vertical and horizontal leg reactions shall be checked against the factored bearing capacity of the leeward leg and sliding capacity of the windward leg. The reactions are determined for a spudcan with ‘fixity’ conditions. The amount of rotational fixity is not directly involved in a checking equation, but is constrained by the yield interaction surface and modifies the forces in both the foundation and structure.

For step 2c, the bearing and sliding checks are performed implicitly through the use of an unfactored yield function described in A.9.3.2.3.

When a level 2a or 2b assessment results in a foundation over-utilization and for all level 2c analyses, a level 3 assessment can be used to calculate the associated displacements. The procedure shall account for the redistribution of forces resulting from the overload and displacement of the spudcan(s). The structural utilizations, overturning utilizations, foundation utilizations and the displacements shall be re-evaluated. 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. If the displacement is significant the effects on the foundation reactions and the structure shall be evaluated and the procedure iterated. Step 3b shall be performed using a structural model including nonlinear soil response.
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:

- bearing capacity (which can exceed preload)
- settlement, including consolidation of trapped soils
- sliding resistance
- drainage paths
- resistance to penetration and retrieval.

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.

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 location 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 could cause damage to the jack-up. The situation can be complicated by the proximity of a fixed structure or wellhead.

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.

9.4.6 Cyclic mobility

Cyclic loads can cause a progressive build-up of pore pressures within the foundation soils and consequent soil strength degradation. The effects can be either local to the soils under the spudcan or over a larger area. Local foundation cyclic loading can be caused by the jack-up response to earthquakes, severe storms, rotating machinery, etc. Earthquakes can cause large scale cyclic loading and result in failure of the soil mass over a large area. Depending on the magnitude of pore pressure developed, cyclic loading can result in large vertical 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.

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), which 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. For further guidance see Annex A.

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 Annex A.

9.4.9 Geohazards

Natural shallow geological features and conditions such as faults, scarps, fluid expulsion features and gas-charged or over-pressurised sediments can pose additional threats to jack-ups that are independent of the foundation loads. 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 like 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 considers the overall geological setting and all the geohazards that can threaten the jack-up or its operations while on location. This work should be conducted and assured by competent geohazard specialists. Further, information is given in Clause A 9.4.8.

9.4.10 Carbonate material

Carbonate materials can exhibit unexpected behaviour and should be addressed with care.

10 Structural response

10.1 Applicability

The response of a jack-up is determined by applying actions to the structural model to determine 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.

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, as well as 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 which shall be checked to determine the adequacy of individual structural components;
- foundation reactions which 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. Additionally, in unusual circumstances, assessments for fatigue resistance, accidental situations, earthquake and abnormal environmental events can be required.

Different methods of analysis can be used for the various limit states to be considered. The methods of analysis include:

- deterministic two-stage analysis, in which the responses of the jack-up are determined by analyzing 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 analyzing multiple combinations of (environmental) actions for each assessment situation. Because of the inherent nonlinearity 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. There are two approaches to incorporating dynamic and foundation response in the analysis: a simplified two-stage approach and a comprehensive nonlinear single-stage approach, see clause 10.4.4.
### Table 10.3-1 Analysis requirements for different assessment situations

<table>
<thead>
<tr>
<th>In-place elevated mode</th>
<th>Deterministic analysis</th>
<th>Stochastic analysis</th>
<th>Ultimate capacity analysis</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>linear</td>
<td>nonlinear</td>
<td>dynamic linear</td>
</tr>
<tr>
<td>Ultimate and serviceability limit states</td>
<td>See clauses 10.5, A.10.5.2 &amp; A.10.5.3</td>
<td>Generally outside the scope of this document. See 10.9</td>
<td></td>
</tr>
<tr>
<td>Fatigue</td>
<td>See clause 11</td>
<td>n/a</td>
<td>See clause 11</td>
</tr>
<tr>
<td>Accidental limit state</td>
<td>Appropriate, but can be unduly conservative</td>
<td>Appropriate, but outside the scope of this document</td>
<td>Appropriate, but can be conservative</td>
</tr>
<tr>
<td>Earthquake</td>
<td>See clauses 10.7 &amp; A.10.7</td>
<td>Appropriate, but outside the scope of this document</td>
<td>Generally outside the scope of this document. See A.10.7.4</td>
</tr>
</tbody>
</table>

### Table 10.3-2 Methods of extreme storm analysis

<table>
<thead>
<tr>
<th></th>
<th>Two-stage approach “Deterministic”</th>
<th>One-stage “Stochastic”</th>
</tr>
</thead>
<tbody>
<tr>
<td>Stage 1</td>
<td>Stage 2</td>
<td></td>
</tr>
<tr>
<td>Determine DAF</td>
<td>Final Response</td>
<td>Need multiple simulations</td>
</tr>
<tr>
<td>SDOF DAF</td>
<td>DAF - “Stochastic”</td>
<td></td>
</tr>
<tr>
<td>Wave/current loads</td>
<td>n/a</td>
<td>Random (superposition of linear components)</td>
</tr>
<tr>
<td>Dynamics</td>
<td>SDOF</td>
<td>High order regular wave</td>
</tr>
<tr>
<td>Wind loads</td>
<td>n/a</td>
<td>Random (linear or higher order)</td>
</tr>
<tr>
<td>Foundation</td>
<td>Linearised</td>
<td>Quasi-static</td>
</tr>
<tr>
<td>Structure</td>
<td>Stiffness from nonlinear structure</td>
<td>Quasi-static</td>
</tr>
<tr>
<td>OUTPUT</td>
<td>DAF</td>
<td>(global) Responses</td>
</tr>
<tr>
<td></td>
<td>DAF</td>
<td>(global) Responses</td>
</tr>
</tbody>
</table>
10.4 Common parameters

10.4.1 General

This subclause presents a description of important parameters that are applicable to all analysis methods.

10.4.2 Natural periods and affecting factors

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 correct 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/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. The natural periods of the jack-up are a function of the static and time-varying response due to nonlinearities in the structural and foundation behaviour. Structural nonlinearities can result from stiffness changes (gap impact, yielding, etc.). Foundation nonlinearities 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 be used in the analysis as it may affect the influence of wave reinforcement and/or cancellation effects.

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

10.4.3 Damping

Contributions to the system damping include foundation damping, hydrodynamic damping and structural damping. Nonlinear behaviour of the foundation and the jacking system also contribute 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 evaluation 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 and inertial loadset, often using linearised 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 nonlinear time-domain stochastic analysis is performed taking into account the elasto-plastic behaviour of the spudcan stiffness.

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 of wave spreading in the storm conditions is available.

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

Seastates with peak period close to the natural period of the jack-up can give larger dynamic amplification resulting in larger responses in smaller seastates. 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 wave and current actions. This behaviour shall be modelled appropriately in the analysis by including the static and dynamic contributions. These effects can be assessed by a deterministic or stochastic analysis procedure. Actions due to fixed load and variable load and wind actions shall be combined with wave and current actions.

A deterministic analysis involves developing static metocean actions and an inertial loadset. The inertial loadset can be developed from either a single-degree-of-freedom (SDOF) method or a stochastic assessment of the wave actions to develop a dynamic amplification factor (DAF). See 10.5.2.

A more detailed stochastic time-domain analysis procedure, in which inertial actions are implicitly included, can 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 to maximise leg and holding system strength utilizations.

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 the extreme wave.
Deterministic responses are normally calculated by time stepping the extreme wave through the structure. The extreme response is determined from:

- the actions due to fixed load and variable load and wind actions;
- the time-dependent, but quasi-static wave/current actions;
- inertial loadset representing dynamic effects.

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

The inertial actions induced by time-varying wave and current actions are approximated by an inertial loadset. The magnitude of the inertial loadset is determined from a dynamic amplification factor (DAF) and the quasi-static wave/current actions. Methods of calculating the DAF include:

- a single degree-of-freedom approximation;
- estimating the ratio of quasi-static and dynamic responses.

When determining DAF’s, P-∆ effects shall be included in both the quasi-static and the dynamic analyses and the contribution of the P-∆ 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 per realization would 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 load, variable load and wind actions are combined with the time-varying wave and 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 stochastic dynamic analyses.

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

The metocean action factors for this analysis method are set to 1.0 according to 8.8.1.3. However, 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 stochastic dynamic analysis. The partial metocean factors 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 according to 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 Annex A.

In all cases, the direction of the moment shall be such as to maximise the maximum utilization checks in way 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 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 critical sea state directions to determine the maximum limit state utilizations.

See also Clauses 9, 12 and 13.

10.6 Fatigue

A fatigue analysis is normally undertaken during the jack-up design phase. For jack-up operations of relatively short duration, when compared with the inspection interval, a fatigue analysis is not required, provided that structural integrity is maintained through a RCS periodic survey or equivalent. For jack-up operations of relatively long duration, see Clause 11.

10.7 Earthquake

An earthquake assessment shall be performed for sites where the ISO 19901-2 seismic zone is 2 or above. An earthquake assessment need not be performed for seismic zone 0. An earthquake assessment should be considered when in seismic zone 1 and any of the following conditions apply:

- Sites with the potential for cyclic mobility (e.g., liquefaction) (ISO 19901-2 soil type f);
- Sites with the potential for unacceptable additional leg penetrations if the preload reactions are exceeded (settlement limits can be smaller when operating adjacent to other structures);
- Jack-ups where the ratio between the individual leg preload reaction at the seabed and the maximum still water operating reaction at the seabed is less than 1.25.

In such cases the structure shall be assessed to the ultimate limit state (ULS) for strength and stiffness, when it is subjected to earthquake actions derived from the uniform hazard spectrum for a return period of 1000 years. Guidance on 1000 year earthquake response spectrum criteria can be found in ISO 19901-2. Under this earthquake the jack-up should sustain little or no damage.

If the jack-up does not satisfy this ULS screening assessment, the alternative assessment methods (see clause 10.9) in combination with ISO 19901-2 shall be used to evaluate compliance 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 does not occur under any of the earthquake events considered, although in some cases considerable structural damage could be sustained.

Since it is not possible to ready the jack-up for an earthquake, it is important to consider all reasonable mass and operating configurations.

NOTE A low mass tends to lead to a shorter natural period, and hence greater amplification. A higher mass results in a longer period, but perhaps greater lateral forces depending on the reduction in the transverse accelerations due to the increased mass.
The assessment model shall consider a realistic range of spudcan-soil modelling that encompasses the uncertainties in foundation stiffness and capacities, see 8.6.3. A pinned spudcan model in general produces an unconservative representation of the earthquake demand on the jack-up.

At locations where cohesionless soil conditions dominate, the possibility of earthquake induced soil liquefaction shall be considered.

10.8 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).

10.9 Alternative analysis methods

10.9.1 Ultimate strength analysis

An ultimate strength analysis is intended to identify the collapse strength of the jack-up structure and foundation under applied actions. 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 capacity based on upper bound or, if necessary, artificially strong, estimates of soil properties. 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.

10.9.2 Types of analysis

Methodology for performing an ultimate strength analysis can be found in ISO 19902. The determination of actions and foundation properties shall be according to this document.

11 Long-term applications

11.1 Applicability

When a jack-up is to be operated on one particular location for longer than the normal special survey period of five years, the site-specific assessment shall be supplemented by the provisions of this clause and RCS requirements.

The specific requirements of the jack-up owner, operator and regulator related to the long-term application shall be investigated.

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 on location;
— a list of modifications to the jack-up which affect the time-varying actions, structural resistance or, fatigue endurance of structural components;

— limitations on the ability to re-level the jack-up and maintain hull elevation e.g. in connection with supported conductors;

— deviations from the standard operating and elevated storm mode configurations given in the marine operations manual;

— metocean data suitable for fatigue assessment, including directionality of wind, waves and current;

— expected accumulation and vertical distribution of marine growth and relevant mitigation procedures;

— geotechnical data required for the assessment of long-term operations; and

— 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 location. 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 design factor ($f_{FD}$). See A.11.3.1 for further details.

The partial action factors used for fatigue analysis may 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 compliance 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 location. 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 which includes the following surveys:

1) special survey prior to deployment on location; and
2) project specific in-service inspection programme (PSIIP) surveys.

The PSIIP required for long-term operations shall be developed based on:

- RCS requirements;
- the jack-up's prior operating and inspection history; and
- 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 sufficient fatigue life, and approval should be 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.

The utilization checks shall be carried out according to the equations in Clause 13.

A suitable method of carrying out the specific calculations required by this clause can be found in A.12. The resistance factors given in Annex B are specifically tied to the calculation methods presented in Annex A and should be re-calibrated if other methods are used.

RCS requirements cover the design, construction, and periodic survey of the jack-up (see Clause 1) and address issues such as material properties, fabrication tolerances, welds, construction details and other parts of the jack-up (e.g. jackhouse and hull structure) and these are not normally addressed here. For example, when the reactions within the fixation system are within the limits set by the manufacturer and approved by the RCS, no additional assessment is required of the hull and jackhouse. Similarly, if the spudcan vertical and rotational reactions are within the limits set by the manufacturer, it is not necessary to check the strength of the leg to spudcan connection.

12.1.2 Truss type legs

The requirements set out in this clause relate to chords and braces. Weld sizes, gusset plates, the strength of joints, etc. are covered by RCS requirements (see Clause 1), 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 this clause are applicable to either tubular or box-type legs but this clause should be supplemented with other documents to address stiffened sections, e.g. References [12.1-1] to [12.1-4].

12.1.4 Fixation system and/or elevating system

Strength of the fixation and/or the elevating system is normally supplied by the manufacturer. The data should represent the unfactored ultimate strength of the system, normally given separately for the vertical and horizontal directions.

12.1.5 Spudcan strength including connection to the leg

The strength of the spudcan is normally supplied by the manufacturer. The data are expected to represent the unfactored ultimate strength of the system, normally given for all applicable vertical and horizontal forces, and for moments about the horizontal axes.

12.1.6 Overview of the assessment procedure

The basic approach consists of the determination of:

- classification of cross sections (12.2);
- section properties (12.3);
- node-to-node Euler amplification of member forces (if not included within the structural analysis) (12.4);
- strength of lattice leg members (tubular members (12.5), and prismatic members (12.6)); and
- strength of joints (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 tubulars and all other cross sections which are called prismatic. Longitudinally reinforced tubulars and tubulars with pin-holes, cut-outs, etc. shall be considered to be prismatic.

12.2.2 Material yield strength

The material yield strength used in the member classification and the calculation of the capacities 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 either a bending moment or an axial force. By classifying cross-sections, the need to explicitly calculate local buckling stresses is avoided.

For prismatic members, the components and cross-sections are classified as plastic, compact, non-compact (or semi-compact) and slender, in order of decreasing capacity. When a cross-section is composed of components of different classes, it shall be classified according to the class of its lowest capacity compression...
components. Slender components within a cross-section may be ignored provided that 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 capability. Compliance with this classification enables a plastic hinge to develop with sufficient rotation capability to allow redistribution of moments to occur within the structure. All plastic sections are inherently compact.

NOTE Compliance with this classification is only relevant when undertaking earthquake, accidental or alternative strength analyses (see 10.7, 10.8 and 10.9). In all other cases the distinction between plastic and compact is irrelevant to the assessment.

Class 2 Compact: Cross-sections with plastic moment capability. Compliance with this classification enables the full plastic moment capability of a cross-section to be developed but local buckling prevents the development of a plastic hinge with sufficient rotation capability to permit plastic assessment.

Class 3 Non-compact (or semi-compact): Cross-sections with full yield moment capability and limited plastic moment capability. Compliance 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 capability.

Class 4 Slender: Cross-sections that buckle locally before yield stress can be achieved. A cross-section is classified as slender if any of the compression components of the cross-section does not comply with the limits for non-compact components.

There is no need for tubular sections to be classified to the same extent as prismatic sections other than to identify those tubulars for which plastic hinge rotation capability is possible (i.e. class 1). This is because the equations presented in A.12.5, strength of tubular members, account for local buckling, whether plastic or elastic.

12.3 Section properties

12.3.1 General

The requirements in this subclause apply to rolled and welded prismatic members comprising one or more components, such as can be found in a chord section of a jack-up leg.

Cross-sectional properties for prismatic members shall be determined as described in this subclause.

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 assessment

The nomenclature and equations required by this subclause 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 assessed moment; this can occur in sections which include components of differing yield strengths. Similarly, for class 4 sections, there is an eccentricity between the full elastic centroid which is used in the response analysis and the centroid of the reduced section 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 analysis, they shall be included in the strength checks. The effective length factors and moment reduction factors \( C_m \) for use in strength checks are listed in Table A.12.4-1. Alternatively, they may be determined using a rational analysis that includes joint flexibility and side-sway.

It is noted 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 requirements given in this subclause ignore the effects of hydrostatic pressure. The validity of this assumption shall be checked for all sealed tubular sections (see Table A.12.5-1).

12.6 Strength of prismatic members

The strength of prismatic members shall be checked for combined axial forces and bending and for shear and torsional shear.

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 location.

The partial action and resistance factors set out in these 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.

NOTE This clause presently addresses the manned non-evacuated condition (L1) only and is based on the criteria given in 6.4. Other levels will be included when they are developed. << e.g. L2 for TRS areas such as GoMex. Ensure this note is up-to-date when we go to FDIS >>.

The criteria for checking the acceptability of a jack-up are discussed in this clause, and include consideration of the following issues:

- structural strength of legs, spudcan, and holding system (13.3, 13.4, and 13.5 respectively);
- hull elevation (13.6);
- leg length reserve (13.7);
- overturning stability (13.8);
- foundation integrity including preload, bearing capacity, sliding displacement, settlement resulting from exceedence of the capacity envelope (13.9); and
- temperature (13.10).

13.1.2 Ultimate limit states (ULS)

The assessment shall ensure that the acceptance criteria are not exceeded in any of the applicable assessment situations; see 5.1, 5.3 and 5.4.

Areas that are often critical on jack-up rigs are the legs at the lower guides, the legs between guides, the pinions and/or rack teeth, the chocks and/or chock supports (if chocks are fitted) and the leg to spudcan connection.

Where foundation fixity exists, the lower parts of the leg shall be checked assuming an upper bound fixity value. Foundation fixity shall only be included in the evaluation of the upper leg when an applicable and detailed foundation study has been made.

Compliance may 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 5 years.

13.2 General formulation

The assessment shall generally follow a partial safety factor format. The partial action factors shall be applied to actions as defined in other clauses and not the action effects.

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.

The utilization of each limit state and assessment situation check shall satisfy the following general form:

\[ U_i = f_\alpha \leq 1.0 \]  \hspace{1cm} (13.2-1)

where

\[ U_i \] is the utilization to one significant decimal place (see also definition of utilization in 3.81)

\[ \alpha_i = \left( \frac{ Action \: Effect \: (A_i) \: due \: to \: Factored \: Action \: (F_i) }{ Factored \: Resistance \: (R) } \right) \]  \hspace{1cm} (13.2-1)

This is a generalized form of the equation, and the form appropriate for each application can differ slightly. In the linear case the function simplifies to a summation. See specific clauses for the particular form of the equation to be used.

The factored action can be generalised as

\[ F_d = A \gamma_f \]  \hspace{1cm} (13.2-3)

where

\[ A \] = Action due to the assessment load case

\[ \gamma_f \] = The specific partial action factor for each type of action

Factored actions shall be determined in accordance with 8.8.

Action effects are determined by following the calculation methodology set out in 10 of this document. The Factored Resistance can be defined as:

\[ R = \frac{Q}{\gamma_R} \]  \hspace{1cm} (13.2-4)

where

\[ Q \] = representative capacity

\[ \gamma_R \] = the resistance factor for each representative capacity

The structural and foundation resistance factors are given in normative Annex B.
13.3 Leg strength

The methodology for undertaking checks on the strength of members is described in Clause 12. In general the formulation given in 13.2 shall be used to assess the utilization of the leg structure.

Resistance factors for assessing the strength of members are given in normative Annex B.

NOTE Reference can be made to the applicable clause in the informative where the relevant factors are given along with the calculation methodology. The table in Normative Annex B contains all the factors to be used in a site-specific analysis, and can be referenced to ensure that the most up to date values are used.

13.4 Spudcan strength

The effects of 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 ultimate strength derived from the manufacturer’s specification using $\gamma_{R,S} = 1.15$. Where limited information is available a rational approach shall be used.

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 When the global response analysis is pinned the forces on the spudcan can be derived from the preload reaction and the soil ultimate moment strength.

13.5 Holding system strength

The forces applied to the holding system due to factored actions shall be checked against the factored ultimate strength derived from the manufacturer's specification using $\gamma_{R,H} = 1.15$. Where limited information is available a rational approach shall be used.

13.6 Hull elevation

A hull elevation resulting in at least 1.5 m clearance between the extreme wave crest elevation and the underside of the hull shall be provided. 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 ISO 19901-1 and A.6.4.2.4) that would 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 crest elevation may be determined accounting for the joint probability of tide, surge and crest elevation.

NOTE Metocean studies after hurricanes Katrina and Rita\textsuperscript{[13.6-1]} have suggested that there exist local wave crest enhancements with a small area of effect. These local effects, over and above the abnormal crest elevation need not be considered when calculating the hull elevation for jack-ups since they do not affect the global response.

The hull elevation is also intended to account for any settlement due to the extreme or abnormal storm event.

NOTE 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

The leg length reserve above the upper guides should reflect the uncertainty in the prediction of leg penetration and account for any settlement. The leg length reserve shall be at least 1.5 m. At locations where there is uncertainty a larger reserve should be available. A larger reserve can also be required due to:
— 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.

### 13.8 Overturning stability

The formulation given in 13.2 shall be used to assess margin of safety against overturning of the jack-up. The utilization shall be calculated based on:

\[
M_{\text{OTM}} = \text{overturning moment due to factored actions } F_d
\]

\[
R_{d\text{OTM}} = \text{the factored stabilizing moment based on the representative stabilizing moment } R_{r\text{OTM}}
\]

The overturning moment shall be calculated from factored actions about the overturning axis in the most critical assessment situation. For independent leg jack-ups the overturning axes shall pass through any two or more spudcan reaction points. The reaction points are given in 8.6.2 and further described in A.8.6.2.

The representative stabilizing moment \( R_{r\text{OTM}} \) shall be calculated about the same axis for the same assessment situation as used to calculate the overturning moment and shall account for the following parameters:

— The stabilizing moments due to the fixed action with the jack-up at the displaced position resulting from the factored actions (e.g. metocean actions).
— The stabilizing moment due to the most onerous combination of minimum action due to variable load and centre of gravity as specified in 5.4.4.
— The stabilizing moments due to seabed foundation fixity. Any stabilizing moments due to fixity shall be calculated in accordance with Clause 9, taking account of any reduction of the moment fixity to comply with the yield surface of the foundation.

Large deflection (\( P-\Delta \)) effects shall be included when computing the overturning utilization. When the overturning moment is calculated from the foundation reactions obtained from a large deflection analysis, the reduction in stabilizing moment due to large deflection effects is implicitly included within the overturning moment. Otherwise the reduction in stabilizing moment from fixed actions and variable load caused by the displacement resulting from the factored actions shall be explicitly included either as an increase in the overturning moment or as a reduction in the stabilizing moment.

The resistance factor on the representative stabilizing moment shall be taken as:

\[
\gamma_{R\text{OTM}} = \text{the resistance factor on representative stabilizing moment } = 1.05
\]

NOTE The overturning check is a traditional benchmark. It serves no other purpose as the foundation checks govern.

### 13.9 Foundation Integrity

#### 13.9.1 Capacity check

The formulation given in 13.2 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 using the formulations given in A.9.3.6. The preload utilization \( U_{S,\text{pl}} \) shall always be computed and reported in accordance with A.9.3.6.2. The following utilizations shall be computed, as required:

\[
U_{S,\text{vhm}} = \text{the foundation bearing capacity utilization}
\]

\[
U_{S,\text{hvm}} = \text{the foundation sliding resistance utilization}
\]
When a yield interaction or continuum foundation model is used these checks are generally not performed. When sliding is not included in the model, a sliding check shall be undertaken in accordance with A.9.3.6.3.

13.9.2 Displacement check

If the reactions on any spudcan due to factored actions exceed the factored bearing capacity and/or the factored sliding resistance discussed in 13.9.1 a further assessment may be performed in order to show that any additional settlements and/or the associated additional structural action effects are within acceptable limits and the hull can be jacked (see A.9.3.6).

NOTE A conservative estimate of the allowable settlement can be derived from the hull inclination limit, if this is specified in the operations manual.

13.10 Temperatures

The 50 year lowest mean daily average air and water temperatures shall be in compliance with the limits given in the operating manual.
Annex A
(informative)

Recommendations and information

NOTE The clauses in this annex provide guidance on the related clause in the body of the document.

A.1 Scope

Although this document does not address the integrity of well conductors, the following reference provides guidance on their assessment: The Institute for Petroleum, Guidelines for the Analysis of Jack-up and fixed Platform Well Conductors (July 2001).

A.2 Normative references

No guidance is offered.

A.3 Terms and definitions

No guidance is offered.

A.4 Symbols

A.4.1 Symbols used in Clause A.1

No guidance is offered.

A.4.2 Symbols used in Clause A.2

No guidance is offered.

A.4.3 Symbols used in Clause A.3

No guidance is offered.

A.4.4 Symbols used in Clause A.4

No guidance is offered.

A.4.5 Symbols used in Clause A.5

No guidance is offered.

A.4.6 Symbols used in Clause A.6

\[ D_1 \] directional spreading function

\[ D_2 \] directional spreading function
$D_3$ directional spreading function

$d$ water depth

$F(\alpha)$ directionality function

$f$ wave frequency

$H_{\text{max}}$ Individual extreme wave height

$H_s$ increased significant wave height to account for wave asymmetry

$H_{\text{srp}}$ significant wave height

$h$ reference depth for wind driven current

$L$ the wavelength of the $H_{\text{max}}, T_{\text{ass}}$ wave in depth $d$, according to regular wave theory

$N$ exponent in formulation for $Z_{\text{ref}}$

$n$ exponent in $D_1$

$S_y$ the smallest spacing between the legs, of 3-legged jack-ups

$S_{\text{PM}}(\omega)$ Pierson-Moskowitz wave spectrum for a seastate

$S_{\text{JS}}(\omega)$ JONSWAP wave spectrum for a seastate

$S_{\eta\eta}(f)$ wave spectral density functions expressed as a function of wave frequency

$S_{\eta\eta}(f, \alpha)$ directional short-crested power density spectrum

$s$ exponent in $D_2$

$T_a$ apparent period of a periodic wave (to an observer in an earth bound reference frame)

$T_{\text{ass}}$ the wave period associated with $H_{\text{max}}$

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

$T_p$ modal or peak period of the spectrum

$T_z$ mean zero-crossing 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)

$V_{\text{ref}}$ the 1 minute sustained wind velocity at elevation $Z_{\text{ref}}$

$V_i$ downwind component of mean spring tidal current

$V_w$ wind generated surface current

$V_Z$ the wind velocity at elevation $Z$ above mean water level

$Z$ elevation above mean water level
A.4.7 Symbols used in Clause A.7

\( z \)  
\( \alpha \)  
\( \gamma \)  
\( \kappa \)  
\( \phi \)  
\( \sigma \)  
\( \psi \)  

distance above still water level (always negative)

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

shape parameter of the peak enhancement factor in the JONSWAP spectrum

kinematics reduction factor

directional spreading factor based on latitude

exponent in \( D_i \)

latitude

\( A \)  
\( A_e \)  
\( A_i \)  
\( A_{Wi} \)  
\( C_A \)  
\( C_{De} \)  
\( C_{Dei} \)  
\( C_{D, D_i} \)  
\( C_{Dpr(\theta)} \)  
\( C_{D00} \)  
\( C_{D01} \)  
\( C_{m, C_{mi}} \)  
\( C_{Mei} \)  
\( C_s \)  
\( D, D_i \)  
\( D_e \)  
\( D_F \)  
\( D_{pr(\theta)} \)  
\( d \)  
\( H_s \)  
\( l_i \)  
area

effective area of leg per unit height

effective area of member or gusset \( i \)

projected area of the block

added mass coefficient

equivalent value of the leg

equivalent value of the drag coefficient of member \( i \)

drag coefficient, drag coefficient of member \( i \)

drag coefficient related to the projected diameter

drag coefficient for a tubular with appropriate roughness

drag coefficient for flow normal to the rack related to projected diameter, \( W \)

inertia coefficient, inertia coefficient of member \( i \)

equivalent value of the inertia coefficient of member \( i \)

shape coefficient

reference diameter, reference diameter of member \( i \)

equivalent diameter of leg

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

projected diameter

still, or undisturbed water depth (positive)

increased significant wave height to account for wave asymmetry

length of member 'i' node to node centre
**ISO/DIS 19905-1**

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

$P_i$  pressure at the centre of the block

$s$  length of one bay, or part of bay considered

$T_n$  first natural period of surge or sway motion

$T_z$  mean zero-crossing period of the water surface elevation in a sea state

$t_m$  marine growth thickness

$W$  projected diameter

$r_n$  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

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

$V_C$  current velocity to be used in the hydrodynamic model

$V_i$  far field (undisturbed) current

$v_n$  relative fluid particle velocity resolved normal to the member axis

$z'$  modified coordinate to be used in particle velocity formulation

$z$  elevation at which the kinematics are required (coordinate measured vertically upward from the still water surface)

$\alpha_i$  angle between flow direction and member axis projected onto a horizontal plane

$\beta_i$  angle defining the member inclination from horizontal

$\Delta F$  wave action per unit length

$\Delta F_{\text{drag}}$  drag action

$\Delta F_{\text{inertia}}$  inertia action

$\lambda$  wave length

$\rho$  mass density of water or air

$\theta$  angle in degrees

$\zeta$  instantaneous water level (same axis system as $z$)

**A.4.8 Symbols used in Clause A.8**
A  axial area of equivalent leg model

$A_s$  effective shear area

$B$  moment amplification factor

$E$  Young's modulus

$F$  applied axial action

\[ F = \frac{(D_b s_t)}{(D_s u)} \]

$F$  axial 'unit' load case

$F_{in}$  inertial loadset

$G$  shear modulus

$I$  second moment of area

$K_{hh}$  horizontal leg/hull connection stiffness

$K_{rh}$  rotational leg/hull connection stiffness

$K_{vh}$  vertical leg/hull connection stiffness

$L$  cantilevered length (from the hull to seabed reaction point)

$M$  applied moment

$P$  pure 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

$\Delta$  axial deflection of cantilever at point of force application

$\Delta_c$  axial end displacements of the detailed leg model

$\Delta_C$  axial end displacements of the combined leg and leg/hull connection model

$\delta$  lateral deflection of cantilever at the point of moment application

$\delta_C$  lateral deflection of the combined leg and leg/hull connection model

$\theta$  slope of cantilever at point of moment application

**A.4.9 Symbols used in Clause A.9**

$A$  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

\[ = \frac{D}{2.5B} \quad \text{for } D < 2.5B \]
\[ = 1,0 \text{ for } D \geq 2,5B \]

\[ a_s \] bearing capacity squeezing factor

\[ B \] effective spudcan diameter at uppermost part of bearing area in contact with the soil (for rectangular footing \( B = \text{width} \))

\[ B_{\text{max}} \] plan diameter of the contact area when the spudcan is fully seated

\[ B_S \] spudcan soil buoyancy due to submerged weight of soil displaced by the spudcan, \( \gamma'V \)

\[ b \] relative strength parameter

\[ = \frac{(D_b s_t)}{(D s_u)} \]

\[ b_s \] bearing capacity squeezing factor

\[ C_H \] horizontal capacity coefficient

\[ = C_{H\text{shallow}} + (C_{H\text{deep}} - C_{H\text{shallow}}) \frac{D}{B} \quad \text{for } D < B \]

\[ = C_{H\text{deep}} \quad \text{for } D \geq B \]

\[ C_{H\text{shallow}} = \frac{(s_{uo}A + (s_{uo} + s_{ul}) A_s)}{Q_{Vnet}} \]

\[ C_{H\text{deep}} = 1,3 \left( \frac{A_s}{A} \right) \]

\[ D \] greatest depth of maximum cross-sectional spudcan bearing area below sea floor

\[ D_b \] depth of backflow (see A.9.3.2.1.4); Infill should not be considered

\[ D_R \] relative density of sand

\[ d \] depth beneath sea floor

\[ d_c \] bearing capacity depth factor

\[ = 1 + 0,2 \left( \frac{D}{B} \right) \leq 1,5 \]

\[ d_{\text{crit}} \] depth at which maximum bearing resistance occurs (layered case)

\[ F_H \] applied factored horizontal force

\[ F_M \] applied factored moment force

\[ F_V \] applied factored vertical force

\[ = V_{st} \quad \text{no backfill} \]

\[ = V_{st} + W_{BF,D} + W_{BF,A} - B_S \quad \text{with backfill} \]

\[ f_1 \] factor used in yield surface equation for embedded spudcans on clay.

\[ f_2 \] factor used in yield surface equation for embedded spudcans on clay.

\[ f_r \] stiffness reduction factor

\[ G \] shear modulus

\[ H \] distance from spudcan maximum bearing area to weak strata below
Limiting depth of cavity that remains open above the spudcan during penetration

Embedment depth to the uppermost part of the spudcan, (if not fully embedded = 0).

Spudcan tip embedment depth.

Rigidity index for normally consolidated clays

Plasticity index

Dimensionless stiffness factor

Active earth pressure coefficient (for \( \kappa_1 = 0 \))

\[
\kappa_a = \tan^2(45 - \phi/2)
\]

Passive earth pressure coefficient

\[
\kappa_p = 1/\kappa_a
\]

Stiffness factors for vertical, horizontal and rotational foundation stiffness respectively

Depth factors for vertical, horizontal and rotational foundation stiffness respectively

Coefficient of punching shear.

Spudcan rotational stiffness reduction parameter

Bearing capacity factor, taken as \( N_c = 6.0 \) for circular footings

Bearing capacity factor

Overconsolidation ratio

Effective overburden pressure at depth, \( D \), of maximum bearing area

Atmospheric pressure

Maximum horizontal foundation capacity.

\[
Q_H = C_H Q_{V_{net}} \quad \text{(clay)}
\]

\[
= 0.075 Q_{V_{net}} B \quad \text{(sand)}
\]

Gross ultimate vertical foundation capacity

Net ultimate vertical foundation capacity

Initial gross ultimate vertical foundation capacity established by preload operations

Ultimate vertical bearing capacity assuming the spudcan bears on the surface of the lower (bottom) clay layer with no backfill

Maximum moment capacity of foundation

\[
Q_M = 0.12 Q_{V_{net}} \quad \text{(sand)}
\]

\[
= (0.1 + 0.05\alpha(1+b/2)) Q_{V_{net}} B \quad \text{(clay)}
\]
$Q_{M_p}$ moment capacity associated with further spudcan penetration under environmental actions (equal to minimum of $Q_{M_p}$ and $Q_{M_v}$).

$Q_{M_p}$ moment capacity when further spudcan penetration leads to fully seated spud conditions.

$Q_{M_v}$ moment capacity under further spudcan penetration, when the actual vertical force is too low to reach fully seated conditions.

$q_0$ surface bearing resistance

$q_{\text{max}}$ maximum bearing resistance (at $d = d_{\text{crit}}$)

$r_f$ failure ratio.

$s_c$ bearing capacity shape factor

$$s_c = 1 + \left(\frac{N_q}{N_c}\right)\left(\frac{B}{L}\right)$$

$s_u$ undrained cohesive shear strength

$s_{u_0}$ undrained cohesive shear strength at deepest depth of maximum bearing area ($D$ below sea floor).

$s_{uH}$ undrained cohesive shear strength at depth of $H_{\text{cav}}$ below sea floor

$s_{um}$ undrained cohesive shear strength at the sea floor

$s_{ub}$ undrained cohesive shear strength - lower clay below spudcan.

$s_{ut}$ undrained cohesive shear strength - upper clay below spudcan

$T$ thickness of weak clay layer underneath spudcan

$V_{Lo}$ preload reaction for the spudcan being considered (this is not the soil capacity, see 3.69)

$V_{st}$ structural vertical reaction beneath the spudcan due to the factored actions as determined from the procedures given in Clause 10, including leg weight and water buoyancy but excluding submerged weight of backfill ($W_{BF,c} + W_{BF,A}$) and spudcan soil buoyancy ($B_S$)

$V_{swl}$ seabed vertical reaction under still water conditions for the spudcan being considered

NOTE << Check whether this should be prefaced "structural", or whether some adjustment is needed now that $Q_v$ has been defined as gross capacity. >>

$W_{BF}$ submerged weight of backfill

$W_{BF,A}$ submerged weight of backfill that occurs after preloading

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

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

$\alpha$ adhesion factor = 1,0 for soft clays, = 0,5 for stiff clays

$\beta$ cone angle

$\delta$ steel/soil friction angle (degrees)

$\gamma'$ submerged (effective) unit weight of soil

$\rho$ rate of increase in shear strength with depth
\( \phi \) angle of internal friction for sand - degrees.

\( \nu \) Poisson's ratio

**A.4.10 Symbols used in Clause A.10**

- **B** equivalent diameter of the spudcan
- **C** subscripts:
  - subscripts:
    - subscripts:
      - subscripts:
        - subscripts:
          - subscripts:
            - subscripts:
              - subscripts:
                - subscripts:
                  - subscripts:
                    - subscripts:
                      - subscripts:
                        - subscripts:
                          - subscripts:
                            - subscripts:
                              - subscripts:
                                - subscripts:
                                  - subscripts:
                                    - subscripts:
                                      - subscripts:
                                        - subscripts:
                                          - subscripts:
                                            - subscripts:
                                              - subscripts:
                                                - subscripts:
                                                  - subscripts:
                                                    - subscripts:
                                                      - subscripts:
                                                        - subscripts:
                                                          - subscripts:
                                                            - subscripts:
                                                              - subscripts:
                                                                - subscripts:
                                                                  - subscripts:
                                                                    - subscripts:
                                                                      - subscripts:
                                                                        - subscripts:
                                                                            - subscripts:
                                                                              - subscripts:
                                                                                - subscripts:
                                                                                  - subscripts:
                                                                                      - subscripts:
                                                                                                        - subscripts:
                                                                                                          - subscripts:
                                                                                                                - subscripts:
                                                                                                                    - subscripts:
                                                                                                                        - subscripts:
                                                                                                                            - subscripts:
                                                                                                                                - subscripts:
                                                                                                                                    - subscripts:
                                                                                                                                        - subscripts:
                                                                                                                                            - subscripts:
                                                                                                                                                - subscripts:
                                                                                                                                                    - subscripts:
                                                                                                                                              - subscripts:
                                                                                                                                              - subscripts:
                                                                                                                                              - subscripts:
                                                                                                                                              - subscripts:
                                                                                                                                              - subscripts:
                                                                                                                                              - subscripts:
                                                                                                                                              - subscripts:
                                                                                                                                              - subscripts:
                                                                                                                                              - subscripts:
                                                                                                                                              - subscripts:
                                                                                                                                              - subscripts:
                                                                                                                                              - subscripts:
                                                                                                                                              - subscripts:
                                                                                                                                              - subscripts:
                                                                                                                                              - subscripts:
                                                                                                                                              - subscripts:
                                                                                                                                              - subscripts:
                                                                                                                                              - subscripts:
                                                                                                                                              - subscripts:
                                                                                                                                              - subscripts:
                                                                                                                                              - subscripts:
                                                                                                                                              - subscripts:
                                                                                                                                              - subscripts:
                                                                                                                                              - subscripts:
                                                                                                                                              - subscripts:
                                                                                                                                              - subscripts:
                                                                                                                                              - subscripts:
                                                                                                                                              - subscripts:
                                                                                                                                              - subscripts:
                                                                                                                                              - subscripts:
                                                                                                                                              - subscripts:
                                                                                                                                              - subscripts:
                                                                                                                                              - subscripts:
                                                                                                                                              - subscripts:
                                                                                                                                              - subscripts:
                                                                                                                                              - subscripts:
                                                                                                                                              - subscripts:
                                                                                                                                              - subscripts:
                                                                                                                                              - subscripts:
                                                                                                                                              - subscripts:
                                                                                                                                              - subscripts:
                                                                                                                                              - subscripts:
                                                                                                                                              - subscripts:
                                                                                                                                              - subscripts:
                                                                                                                                              - subscripts:
                                                                                                                                              - subscripts:
                                                                                                                                              - subscripts:
                                                                                                                                              - subscripts:
                                                                                                                                              - subscripts:
                                                                                                                                              - subscripts:
                                                                                                                                              - subscripts:
                                                                                                                                              - subscripts:
                                                                                                                                              - subscripts:
                                                                                                                                              - subscripts:
                                                                                                                                              - subscripts:
                                                                                                                                              - subscripts:
                                                                                                                                              - subscripts:
                                                                                                                                              - subscripts:
                                                                                                                                              - subscripts:
                                                                                                                                              - subscripts:
                                                                                                                                              - subscripts:
                                                                                                                                              - subscripts:
                                                                                                                                              - subscripts:
                                                                                                                                              - subscripts:
                                                                                                                                              - subscripts:
                                                                                                                                              - subscripts:
                                                                                                                                              - subscripts:
                                                                                                                                              - subscripts:
                                                                                                                                              - subscripts:
                                                                                                                                              - subscripts:
                                                                                                                                              - subscripts:
                                                                                                                                              - subscripts:
                                                                                                                                              - subscripts:
                                                                                                                                              - subscripts:
                                                                                                                                              - subscripts:
                                                                                                                                              - subscripts:
                                                                                                                                              - subscripts:
                                                                                                                                              - subscripts:
                                                                                                                                              - subscripts:
                                                                                                                                              - subscripts:
                                                                                                                                              - subscripts:
                                                                                                                                              - subscripts:
                                                                                                                                              - subscripts:
                                                                                                                                              - subscripts:
                                                                                                                                              - subscripts:
                                                                                                                                              - subscripts:
                                                                                                                                              - subscripts:
                                                                                                                                              - subscripts:
                                                                                                                                              - subscripts:
                                                                                                                                              - subscripts:
                                                                                                                                              - subscripts:
                                                                                                                                              - subscripts:
                                                                                                                                              - subscripts:
                                                                                                                                              - subscripts:
                                                                                                                                              - subscripts:
                                                                                                                                              - subscripts:
                                                                                                                                              - subscripts:
                                                                                                                                              - subscripts:
                                                                                                                                              - subscripts:
                                                                                                                                              - subscripts:
                                                                                                                                              - subscripts:
                                                                                                                                              - subscripts:
                                                                                                                                              - subscripts:
                                                                                                                                              - subscripts:
                                                                                                                                              - subscripts:
                                                                                                                                              - subscripts:
                                                                                                                                              - subscripts:
                                                                                                                                              - subscripts:
                                                                                                                                              - subscripts:
                                                                                                                                              - subscripts:
                                                                                                                                              - subscripts:
                                                                                                                                              - subscripts:
                                                                                                                                              - subscripts:
                                                                                                                                              - subscripts:
                                                                                                                                              - subscripts:
                                                                                                                                              - subscripts:
                                                                                                                                              - subscripts:
                                                                                                                                              - subscripts:
                                                                                                                                              - subscripts:
                                                                                                                                              - subscripts:
                                                                                                                                              - subscripts:
                                                                                                                                              - subscripts:
                                                                                                                                              - subscripts:
                                                                                                                                              - subscripts:
                                                                                                                                              - subscripts:
                                                                                                                                              - subscripts:
                                                                                                                                              - subscripts:
                                                                                                                                              - subscripts:
                                                                                                                                              - subscripts:
                                                                                                                                              - subscripts:
                                                                                                                                              - subscripts:
                                                                                                                                              - subscripts:
                                                                                                                                              - subscripts:
                                                                                                                                              - subscripts:
                                                                                                                                              - subscripts:
                                                                                                                                              - subscripts:
                                                                                                                                              - subscripts:
                                                                                                                                              - subscripts:
                                                                                                                                              - subscripts:
                                                                                                                                              - subscripts:
                                                                                                                                              - subscripts:
                                                                                                                                              - subscripts:
                                                                                                                                              - subscripts:
                                                                                                                                              - subscripts:
                                                                                                                                              - subscripts:
                                                                                                                                              - subscripts:
                                                                                                                                              - subscripts:
                                                                                                                                              - subscripts:
                                                                                                                                              - subscripts:
                                                                                                                                              - subscripts:
                                                                                                                                              - subscripts:
                                                                                                                                              - subscripts:
                                                                                                                                              - subscripts:
                                                                                                                                              - subscripts:
                                                                                                                                              - subscripts:
                                                                                                                                              - subscripts:
                                                                                                                                              - subscripts:
                                                                                                                                              - subscripts:
                                                                                                                                              - subscripts:
                                                                                                                                              - subscripts:
                                                                                                                                              - subscripts:
                                                                                                                                              - subscripts:
                                                                                                                                              - subscripts:
                                                                                                                                              - subscripts:
                                                                                                                                              - subscripts:
                                                                                                                                              - subscripts:
                                                                                                                                              - subscripts:
                                                                                                                                              - subscripts:
                                                                                                                                              - subscripts:
                                                                                                                                              - subscripts:
                                                                                                                                              - subscripts:
                                                                                                                                              - subscripts:
                                                                                                                                              - subscripts:
                                                                                                                                              - subscripts:
                                                                                                                                              - subscripts:
                                                                                                                                              - subscripts:
                                                                                                                                              - subscripts:
                                                                                                                                              - subscripts:
                                                                                                                                              - subscripts:
                                                                                                                                              - subscripts:
$\rho$  total, saturated, (mass) density of the foundation soil

$\zeta$  damping ratio or fraction of critical damping

$\zeta_{rd}$  radiation modal damping ratio to account for spudcan vertical motion

$\omega_n$  natural frequency (radians)

**A.4.11 Symbols used in Clause A.11**

$D_e$  calculated existing fatigue damage prior to arriving at location

$D_s$  calculated fatigue damage during planned operations on location

$f_{FD,s}$  fatigue design factor, this is generally determined from the applicable table below

$f_{FD,e}$  $f_{FD,s}$, but not larger than 2, provided the detail has been inspected thoroughly before the long term application

**A.4.12 Symbols used in Clause A.12**

$A$  gross cross-sectional area

$A_{ec}$  total effective area of a section in compression

$A_c$  cross-sectional area to be used in the assessment of a member in compression

$A_{eff,i}$  effective area of a compressed component

$A_i$  cross-sectional area of a semi-compact section

$A_i$  cross-sectional area of the $i$th component comprising the structural member

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

$A_p$  fully plastic effective cross-sectional area of a prismatic section

$A_t$  cross-sectional area to be used in the assessment of a member in tension

$A_v$  effective shear area of prismatic in the direction being considered

$B$  member moment amplification factor for the axis under consideration

$b$  width of the wall a component forming the closed perimeter of a section

$b$  effective width of a component

$C_m$  moment reduction factor

$C_s$  critical elastic buckling coefficient

$D$  outside diameter

$D$  depth of cross section

$d$  effective depth of a component

$d$  effective head of water
\( d_i \) distance between the centroid of the \( i^{th} \) component and the plastic neutral axis

\( E \) Young's modulus (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

\( F_{y_{\text{eff}}} \) effective yield strength of a prismatic member cross-section, in stress units

\( F_{yi} \) yield strength of the \( i^{th} \) component of a prismatic member cross-section, in stress units

\( F_{y_{\text{min}}} \) minimum yield strength, \( F_{yi} \), of all components in a prismatic member cross-section, in stress units

\( F_{y_{\text{ltb}}} \) yield strength, \( F_y \), of the material that first yields when bending about the minor axis

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

\( I \) second moment of area of a tubular

\( I_e \) effective second moment of area of a prismatic section

\( I_i \) second moment of a semi-compact prismatic section

\( I_p \) polar moment of inertia

\( I_y \) major axis second moment of area of the gross cross-section

\( I_z \) minor axis second moment of area of the gross cross-section

\( J \) torsion constant

\( K \) effective length factor

\( L \) unbraced length of member for plane of flexural buckling

\( L \) unbraced length in y or z direction measured between centre-lines

\( L_b \) effective length of beam-column between supports

\( L_{lb} \) laterally unbraced length; i.e. length between points which are either braced against lateral displacement of the compression flange or braced against twist of the cross section in addition to lateral support

\( L_p \) limiting plastic length

\( L_t \) limiting unbraced length for inelastic torsional bucking

\( M_b \) representative bending moment strength

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

\( M_{p} \) plastic moment strength
$M_u$  moment in a member determined in an analysis which includes global P-Δ effects

$M_{u_{xy}}$, $M_{u_{xz}}$  amplified bending moments about member y- and z-axes respectively due to factored actions

$M_{u_{xy}}$, $M_{u_{yz}}$  corrected bending moments about member y- and z-axes respectively due to factored actions

$M_{u_{xy}}$, $M_{u_{xz}}$  bending moments about member y- and z-axes respectively due to factored actions determined in an analysis which includes global P-Δ effects

$M_{ua}$  amplified moment

$M_{ue}$  corrected effective moment

$P_a$  representative axial compressive strength

$P_E$  Euler buckling capacity

$P_n$  representative compression strength based on local strength

$P_p$  representative axial strength for the beam column check

$P_{pl}$  compressive axial strength of prismatic members

$P_t$  axial tensile strength of prismatic members

$P_u$  applied axial force

$P_{ut}$  axial tensile force

$P_{uc}$  axial compressive force

$P_v$  representative tubular shear strength

$P_{vy}$, $P_{vz}$  representative prismatic shear strength in the local y and z directions

$P_{xe}$  representative elastic local buckling strength

$P_{yc}$  representative local buckling strength

$p$  depth of penetration below sea floor (zero if above sea floor)

$r_z$  radius of gyration to be used for lateral-torsional buckling considerations

$r$  radius of gyration

$r$  maximum distance from centroid to an extreme fibre

$r_z$  radius of gyration about the minor axis

$S_e$  reduced section modulus

$S_l$  section modulus of a semi-compact section

$S_{y_{y}}$, $S_{z_{z}}$  section moduli to be used in the assessment of a member in flexure

$T_u$  torsional moment due to factored actions

$T_v$  representative torsional strength
$t$  wall thickness of a tubular
$t$  thickness of a wall of a component forming the closed perimeter of a section
$t_f$  thickness of a flange component
$t_w$  thickness of a web component
$V$  beam shear due to factored actions
$V_y, V_z$  beam shear due to factored actions in the local y and z directions
$Z_p$  fully plastic (effective) section modulus
$\alpha$  factor that varies depending on the loading
$\gamma'$  submerged unit weight of the soil
$\gamma'_{R,Pa}$  partial resistance factor for prismatic axial loading
$\gamma'_{R,Pb}$  partial resistance factor for prismatic bending
$\gamma'_{R,Pcl}$  partial resistance factor for prismatic local axial compressive strength
$\gamma'_{R,Pl}$  partial resistance factor for prismatic axial tension
$\gamma'_{R,Pc}$  partial resistance factor for prismatic axial compressive strength
$\gamma'_{R,Pv}$  partial resistance factor for beam shear strength
$\gamma'_{R,Tb}$  partial resistance factor for tubular bending
$\gamma'_{R,Tl}$  partial resistance factor for tubular axial tension
$\gamma'_{R,Tc}$  partial resistance factor for tubular axial compressive strength
$\gamma'_{R,Tv}$  partial resistance factor for tubular beam shear strength
$k$  buckling coefficient
$\lambda$  column slenderness parameter
$\lambda$  $b/t$ or $2R/t$ as applicable for component $h$
$\lambda_c$  prismatic column slenderness parameter
$\lambda_t$  elastic plate slenderness parameter
$\lambda_p$  plate slenderness parameter
$\lambda_{lim}$  limiting plate slenderness ratio
$\lambda_{po}$  plate slenderness ratio coefficient
$\eta$  exponent for biaxial bending, a constant dependent on the prismatic member cross section geometry
\( \rho \) reduction coefficient
\( \rho_w \) weight density of water
\( \sigma_1 \) compressive stress if \( \sigma_2 \) tensile or larger compressive stress if \( \sigma_2 \) compressive
\( \sigma_2 \) tensile stress if \( \sigma_2 \) tensile or smaller compressive stress if \( \sigma_2 \) compressive
\( \psi \) compression to bending stress ratio
\( \theta \) angle between the shear force direction being considered and the larger dimension of the cross-section of component \( i \)

A.5 Overall considerations

No guidance is offered.

A.6 Data to be assembled for each site

A.6.1 Scope

No guidance is offered.

A.6.2 Rig 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 the manned jack-ups 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 manned evacuated, it is assumed that reliable forecasting of the extreme storm event is feasible, evacuation plans are established and documented, and time and resources are available to safely evacuate all personnel from 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 to safely evacuate the jack-up, 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 defined as one which forms locally, and due to speed of formation and proximity to infrastructure at time of formation, may not allow sufficient time to evacuate manned facilities. The population of storms used to derive the sudden hurricane at a given site may therefore be defined in terms of the time horizon required to evacuate the site. For manned-evacuated jack-ups utilized in these circumstances, consideration should be given to the use of a 50-yr return period “sudden hurricane” or “sudden storm” event. An unmanned jack-up may also be assessed using these criteria.
Partial factors for each of these options are presented in 5.5.4.

Site-specific data, if available, should be used for the assessment as regional data may not take account of local variations.

Where there is sufficient evidence that any of the metocean actions at the site are directional, it may be possible to align the jack-up on the most advantageous heading.

A.6.4.2 Waves

A.6.4.2.1 General

The extreme wave environment should be computed according to the following sub-clauses. 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). 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.

A.6.4.2.2 Extreme wave height

The wave height information for a specific site can be expressed in terms of $H_{\text{max}}$, the individual extreme wave height for the return period, or the significant wave height $H_{\text{srp}}$. The relationship between $H_{\text{srp}}$ and $H_{\text{max}}$ should be determined accounting for the effects of storms (3 hours minimum) and for the additional probability of other return period storms, see ISO TR 19905-2 6.4.2.2. This relationship depends on the regional and site-specific conditions, however $H_{\text{srp}}$ may usually be determined from $H_{\text{max}}$ using the generally accepted relationship for non-cyclonic areas:

$$H_{\text{max}} = 1.86 \, H_{\text{srp}} \quad \text{(A.6.4-1)}$$

For cyclonic areas the recommended relationship is:

$$H_{\text{max}} = 1.75 \, H_{\text{srp}} \quad \text{(A.6.4-2)}$$

The wave action can be computed stochastically (through a time domain approach) or deterministically (through an individual maximum wave approach). The two approaches are discussed in A.6.4.2.5 to A.6.2.4.8 and A.6.4.2.3, (see also ISO TR 19905-2 6.4.2.2) and should be used only in conjunction with the associated kinematics modelling recommended in A.7 and the hydrodynamic coefficients given in A.7.3.

A.6.4.2.3 Deterministic waves

For deterministic/regular calculation of wave action it is appropriate to apply a kinematics factor to the horizontal and vertical velocities and accelerations in order to obtain realistic action estimates for the extreme storm event. This factor accounts for the need to ensure that both the deterministic/regular wave action calculation and 3 hour simulation produce statistically comparable results (i.e. both target the MPME response in the 50 year storm). In addition, the factor may be considered to take some account of wave spreading and the conservatism of deterministic/regular wave kinematics. The effect of the factor can be achieved either by scaling of wave kinematics (preferred) or a wave height reduction, but not both.

The kinematics reduction factor, $\kappa$, to be applied to the kinematics obtained from $H_{\text{max}}$ can be determined from:

$$\kappa = \phi$$

where
\( \phi \) = site specific value from metocean data, or for open water conditions directional spreading factor based on latitude \( \psi \) in degrees and type of storm or region, as defined in ISO 19901-1 Clause A.8.7.2:

- Low latitude monsoons typically \(| \psi | < 15^\circ \) 0.88
- Tropical cyclones below approximately 40° 0.87
- Extratropical storms for the range of latitudes 36° < \(| \psi | < 72^\circ \) \(1.0193 - 0.00208 | \psi |\)

Alternatively, the following formulation can be used, Reference [A.6.4-1]:

\[
\kappa = 0.824 \phi + 0.426 \phi^2 - 0.043 \left( \frac{S_y}{L} \right) \phi - 1.450 \left( \frac{S_y}{L} \right)^2 \phi
- 0.800 \left( \frac{d}{L} \right) \phi + 0.658 \left( \frac{d}{L} \right)^2 - 0.640 \left( \frac{H_{\text{max}}}{d} \right) + 1.303 \left( \frac{H_{\text{max}}}{d} \right)^2 \phi^2
\]

(A.6.4-3)

and subject to:

\[
0.08 \leq \left( \frac{S_y}{L} \right) \leq 0.43 \quad \text{(A.6.4-4a)}
\]

\[
0.14 \leq \left( \frac{d}{L} \right) \leq 0.76 \quad \text{(A.6.4-4b)}
\]

\[
0.07 \leq \left( \frac{H_{\text{max}}}{d} \right) \leq 0.58 \quad \text{(A.6.4-4c)}
\]

Where

- \( S_y \) = the smallest spacing between the legs of 3-legged jack-ups
- \( d \) = the water depth,
- \( H_{\text{max}} \) = maximum wave height
- \( T_{\text{ass}} \) = the wave period associated with \( H_{\text{max}} \)
- \( L \) = the wavelength of the \( H_{\text{max}}, T_{\text{ass}} \) wave in depth \( d \), according to the regular wave theory that is being used.

The limiting values \( \frac{S_y}{L} = 0.43 \), \( \frac{d}{L} = 0.76 \) and \( \frac{H_{\text{max}}}{d} = 0.07 \) may be applied for calculation of \( \kappa \) in Equation A.6.4-3 in case these bounds are transgressed.

The upper bound on \( \kappa \) may be taken as \( \phi \).

Where the parameters limited by Equations A.6.4-4 extend outside these limits, \( \kappa = \phi \) can be used.

The kinematics reduction factor formulation was developed for 3-legged drag-dominated jack-ups. Caution should be exercised if it is to be applied to other cases. The formulation should not be applied for the small wave conditions that dominate in FLS assessment and it is noted that such cases are likely to be outside the limits of applicability, however \( \kappa = \phi \) can be applied to such cases.
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.

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 wave period $T_{\text{ass}}$, in seconds, associated with the maximum wave can be considered. 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, Metocean Design and Operation Conditions, Clauses 5.2 and 8.3. Unless site-specific information indicates otherwise $T_{\text{ass}}$ is normally between the following limits:

$$3.44 \sqrt{(H_{\text{srp}})} < T_{\text{ass}} < 4.42 \sqrt{(H_{\text{srp}})}$$  \hspace{1cm} (A.6.4-5)

where $H_{\text{srp}}$ is the return period extreme significant wave height in meters.

**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, $H_{\text{max}}$ in A.6.4.2.2, and the appropriate deterministic wave theory in A.7.4.3.3.1.

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/extreme wave crest should be calculated based on storm statistics and according to principles as described in ISO 19901-1 A.8.8. Examples for the regional application of these principles may be found in Reference [A.6.4-2], or for general application in Reference [A.6.4-3].

If a wave height reduction factor is used in a deterministic wave analysis to represent 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, $S_{\text{PM}}(\omega)$, and the JONSWAP, $S_{\text{JS}}(\omega)$, wave spectra are presented in ISO 19901-1 A.8.6. 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 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.7.4.3, 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, $d$ in meters, from:

$$H_s = [1 + (10 H_{\text{srp}} / T_p^2) e^{-d/25}] H_{\text{srp}}$$

(A.6.4-6)

and should be used with the wave kinematics model described in A.7.4.3.

### A.6.4.2.7 Peak and zero-upcrossing periods

When undertaking a stochastic analysis (either for a one stage analysis or for determining a DAF) it is necessary to either consider a range of wave periods or a suitable wave spectrum which contains sufficient breadth of peak to capture the dynamic characteristics. Information on the range of periods to use is given in this clause, however, to avoid the need for analyses of several wave periods, a practical alternative is to use a 2 parameter spectrum with $\gamma = 1.0$, in combination with the site-specific most probable peak period.

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 an expression for zero-upcrossing period $T_z$, related to $H_{\text{srp}}$ in meters, as follows:

$$3.2 \sqrt{(H_{\text{srp}})} < T_z < 3.6 \sqrt{(H_{\text{srp}})}$$

(A.6.4-7)

However in shallow water the wave steepness can increase to 1/12 or more, leading to a zero-upcrossing period $T_z$ as low as $2.8 \sqrt{(H_{\text{srp}})}$. This is because the wave height increases and wave length decreases for a given $T_z$.

When considering a JONSWAP spectrum, the peak enhancement factor $\gamma$ varies between 1 and 7 with a most probable average value of 3.3. There is no firm relationship between $\gamma$, $H_s$ and $T_p$. Relationships between variables for different $\gamma$ according to Reference [A.6.4-4] are as follows:

<table>
<thead>
<tr>
<th>$\gamma$</th>
<th>$T_p / T_z$</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1.406</td>
</tr>
<tr>
<td>2</td>
<td>1.339</td>
</tr>
<tr>
<td>3</td>
<td>1.295</td>
</tr>
<tr>
<td>3.3</td>
<td>1.286</td>
</tr>
<tr>
<td>4</td>
<td>1.260</td>
</tr>
<tr>
<td>5</td>
<td>1.241</td>
</tr>
<tr>
<td>6</td>
<td>1.221</td>
</tr>
<tr>
<td>7</td>
<td>1.205</td>
</tr>
</tbody>
</table>

Unless site specific information indicates otherwise $\gamma = 3.3$ can be used.

If a JONSWAP spectrum is applied, the response analysis should consider a range of periods associated with $H_{\text{srp}}$ based on the most probable value of $T_p$ 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.
A.6.4.2.8 Short-crested stochastic waves

For stochastic/random wave action calculations, the short-crestedness of waves (i.e. the angular distribution of wave energy about the dominant direction) may be accounted for 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 \( F(\alpha) \), as follows:

\[
S_{\eta\eta}(f, \alpha) = S_{\eta\eta}(f).F(\alpha)
\]

(A.6.4-8)

where

\( \alpha \) = angle between direction of elementary wave trains and dominant direction of the short-crested waves.

\( S_{\eta\eta}(f, \alpha) \) = directional short-crested power density spectrum.

\( F(\alpha) \) = directionality function.

Directionality functions for extreme and fatigue analyses can be found in ISO 19901-1 A.8.7 and ISO TR 19905-2 6.4.2.8. When referring to the formulations in ISO 19901-1 A.8.7 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 A.8.7 then directional spreading function \( D_1 \) with \( n = 8 \) gives good agreement with the formulation in ISO TR 19905-2 6.4.2.8. For directional spreading function \( D_2 \) with \( s = 17 \) and for directional spreading function \( D_3 \) with \( \sigma = 0.34 \) there is good agreement with the formulation in ISO TR 19905-2 6.4.2.8.

The modelling of short-crested stochastic waves in the present clause 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 may 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 wave height/period combinations at the assessment return period probability level.

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 zero-upcrossing period scatter diagram or as a table of representative sea states.

A.6.4.3 Current

Currents 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 down-wind 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:

\[ V_C = V_t + V_s + (V_w - V_s) [(h+z)/h] \quad \text{for } |z| \leq h \text{ and } V_s < V_w \]  
(A.6.4-9a)

\[ V_C = V_t + V_s \quad \text{for } |z| > h \text{ or } V_s \leq V_w \]  
(A.6.4-9b)

where

- \( V_C \) = current velocity as a function of \( z \). Note that a reduction may be applicable according to A.7.4.4.
- \( V_t \) = downwind component of mean spring tidal current.
- \( V_s \) = downwind component of associated surge current (excluding wind driven component).
- \( V_w \) = wind generated surface current. In the absence of other data this may conservatively be taken as 2.6% of the 1 minute sustained wind velocity at 10 m.
- \( h \) = reference depth for wind driven current. In the absence of other data \( h \) should be taken as 10 m.
- \( z \) = distance above still water level (SWL) under consideration (always negative).

Alternative formulations are provided in ISO 19901-1 Clause A.9.3. Comparisons of combined current and wave forces in ISO TR 19905-2 Clause 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.

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.

Normally for a fatigue analysis, current may be neglected.
A.6.4.4 Water depths

The mean sea level (MSL) related to the sea floor is defined in clause 3.

The still water levels (SWL) used for the assessment of the location should be determined and related to lowest astronomical tide (LAT). The relationship between LAT and Chart Datum is discussed in ISO TR 19905-2 6.4.4.

Different extreme water levels are needed for the ULS assessment and hull elevation determination. Unless reliable joint probability data are available, the extreme still water level (SWL), expressed as a height above LAT can be taken as:

Mean high water spring tide (MHWS) + relevant return period extreme storm surge

When lower water levels are more onerous for action calculations, the minimum still water level (SWL) expressed as a height above LAT should be taken as:

Mean Low Water Spring Tide (MLWS) + 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 minute sustained wind, related to a reference level of 10 m above mean sea level.

The wind velocity profile may be defined by a logarithmic function according to ISO 19901-1, or approximated by a power law see A.6.4.6.2. A comparison of wind forces shows that the below power law is slightly more conservative than the ISO 19901-1 logarithmic profile, see ISO TR 19905-2 Clause 6.4.6.1. Typically the average effect is in the range of 7 % for a 1 minute average wind speed of 10 m/sec at 10 m above sea level, and 2 % for a 1 minute average wind speed of 40 m/sec.

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.

Formations for calculation of wind actions are given in A.7.3.4.

A.6.4.6.2 Wind profile

The expression for the vertical profile of the mean wind velocity in the form of a power law is:

\[ V_Z = V_{ref} \left( \frac{Z}{Z_{ref}} \right)^{1/N} \] (A.6.4-10)

where

\[ V_Z = \text{the wind velocity at elevation } Z \text{ above mean water level.} \]

\[ V_{ref} = \text{the 1 minute sustained wind velocity at elevation } Z_{ref} \text{ (normally 10 m above MSL).} \]
\[ N = 10 \] unless site-specific data indicate that an alternative value of \( N \) is appropriate.

### 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 location and the foundation stability. The area covered should be sufficiently large to encompass any stand-off location; normally a 1 km x 1 km square is sufficient. Aspects, which should be investigated, are shown in Table A.6.5-1 and are discussed in more detail in the referenced sections. The information obtained from the surveys and investigations set out in Clauses A.6.5.1.1 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, it may be possible to reduce the recommendations set out below by use of information obtained from other surveys or activities in the area.

**NOTE** Experience of prior jack-up operations at the same site might be used provided the previous bearing pressures exceed those for the present operation by an adequate margin.

<table>
<thead>
<tr>
<th>RISK</th>
<th>METHODS FOR EVALUATION &amp; PREVENTION</th>
<th>CLAUSE</th>
</tr>
</thead>
<tbody>
<tr>
<td>Installation Problems</td>
<td>Bathymetric survey</td>
<td>A.6.5.1.1</td>
</tr>
<tr>
<td></td>
<td>Sea floor survey</td>
<td>A.6.5.2.2</td>
</tr>
<tr>
<td>Punch-through</td>
<td>Geophysical survey</td>
<td>A.6.5.1.3</td>
</tr>
<tr>
<td></td>
<td>Soil sampling and other geotechnical testing and analysis</td>
<td>A.6.5.1.4, A.9.3.2.7</td>
</tr>
<tr>
<td>Settlement/Bearing failure</td>
<td>Geophysical survey</td>
<td>A.6.5.1.3</td>
</tr>
<tr>
<td></td>
<td>Soil sampling and other geotechnical testing and analysis</td>
<td>A.6.5.1.4</td>
</tr>
<tr>
<td></td>
<td>Ensure adequate jack-up preload capability</td>
<td>A.9.3.3</td>
</tr>
<tr>
<td>Sliding failure</td>
<td>Geophysical survey</td>
<td>A.6.5.1.3</td>
</tr>
<tr>
<td></td>
<td>Soil sampling and other geotechnical testing and analysis</td>
<td>A.6.5.1.4</td>
</tr>
<tr>
<td></td>
<td>Increase vertical spudcan reaction</td>
<td>A.9.3.3.2</td>
</tr>
<tr>
<td></td>
<td>Modify the spudcans</td>
<td></td>
</tr>
<tr>
<td>Scour</td>
<td>Bathymetric and sea floor survey (identify sand waves)</td>
<td>A.6.5.1.1</td>
</tr>
<tr>
<td></td>
<td>Surface soil samples and sea floor currents</td>
<td>A.6.5.1.2</td>
</tr>
<tr>
<td></td>
<td>Inspect spudcan foundation regularly</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Install scour protection (gravel bag/artificial seaweed) when anticipated</td>
<td>9.4.7</td>
</tr>
<tr>
<td>Geohazards (mudslides, mud</td>
<td>Sea floor survey</td>
<td>A.6.5.1.2</td>
</tr>
<tr>
<td>volcanoes etc)</td>
<td>Geophysical survey</td>
<td>A.6.5.1.3</td>
</tr>
<tr>
<td></td>
<td>Soil sampling and other geotechnical testing and analysis</td>
<td>A.6.5.1.4</td>
</tr>
<tr>
<td>Gas pockets/Shallow gas</td>
<td>Geophysical survey</td>
<td>A.6.5.1.3</td>
</tr>
</tbody>
</table>
### Risk Methods for Evaluation & Prevention

<table>
<thead>
<tr>
<th>RISK</th>
<th>METHODS FOR EVALUATION &amp; PREVENTION</th>
<th>CLAUSE</th>
</tr>
</thead>
<tbody>
<tr>
<td>Faults</td>
<td>Geophysical survey</td>
<td>A.6.5.1.3</td>
</tr>
<tr>
<td>Metal or other object, sunken wreck, anchors, pipelines etc.</td>
<td>Magnetometer and sea floor survey</td>
<td>A.6.5.1.2</td>
</tr>
<tr>
<td>Local holes (depressions) in sea floor, reefs, pinnacle rocks, non-metallic structures or wooden wreck</td>
<td>Sea floor survey Diver/ROV inspection</td>
<td>A.6.5.1.2</td>
</tr>
<tr>
<td>Leg extraction difficulties</td>
<td>Soil sampling and other geotechnical testing and analysis</td>
<td>A.6.5.1.4</td>
</tr>
<tr>
<td></td>
<td>Consider change in spudcans Jetting/Airlifting</td>
<td>A.6.5.1.4</td>
</tr>
<tr>
<td>Eccentric spudcan reactions</td>
<td>Geophysical survey</td>
<td>A.6.5.1.1, A.6.5.1.2</td>
</tr>
<tr>
<td></td>
<td>Geophysical survey (buried channels or footprints)</td>
<td>A.6.5.1.3</td>
</tr>
<tr>
<td></td>
<td>Soil sampling and other geotechnical testing and analysis</td>
<td>A.6.5.1.4 A.9.3 ???</td>
</tr>
<tr>
<td></td>
<td>Seabed modification</td>
<td></td>
</tr>
<tr>
<td>Seabed slope</td>
<td>Geophysical survey</td>
<td>A.6.5.1.1, A.6.5.1.2</td>
</tr>
<tr>
<td></td>
<td>Seabed modification</td>
<td></td>
</tr>
<tr>
<td>Footprints of previous jack-ups</td>
<td>Evaluate site records</td>
<td>A.6.5.1.1 A.6.5.1.2</td>
</tr>
<tr>
<td></td>
<td>Prescribed installation procedures</td>
<td>9.4.1</td>
</tr>
<tr>
<td></td>
<td>Consider filling/modification of holes as necessary</td>
<td>9.4.1</td>
</tr>
</tbody>
</table>

#### A.6.5.1.2 Bathymetric survey

An appropriate bathymetric survey should be supplied for an area approximately 1 kilometre square centred on the proposed location within the site. Line spacing of the survey should typically be not greater than 100 metres x 250 metres over the survey area. Interlining is to be performed within an area 200 metres x 200 metres centred on the proposed location within the site. Interlining should have spacing less than 25 metres x 50 metres. Such surveys are normally carried out using acoustic reflection systems.

#### A.6.5.1.3 Sea floor survey

The sea floor should be surveyed using sidescan sonar or high-resolution multibeam echosounder techniques and should be of sufficient quality to identify obstructions and sea floor features and should cover the immediate area (normally a 1 km square) of the intended location. 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 location. In some cases an ROV or diver’s 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.

A.6.5.1.4 Shallow seismic survey

A shallow seismic survey uses high resolution acoustic reflection techniques to:

- determine near surface soil stratigraphy.
- reveal the presence of shallow gas concentrations and other geohazards.

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 kilometre square area centred on the proposed location. Line spacing of the survey should typically be not greater than 100 metres x 250 metres over the survey area. The survey report should include at least two vertical cross-sections passing through the proposed location showing all the relevant reflectors and allied geological information. The equipment used should be capable of identifying reflectors of 0.5 m and thicker to a depth equal to the greater of 30 metres 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:

- nearby geotechnical data is not available;
- the shallow seismic survey cannot be interpreted with any certainty;
- significant layering of the strata is indicated; or
- the site is known to be potentially hazardous.

A.6.5.1.5.2 Soil investigation and testing

A geotechnical investigation should comprise a minimum of one borehole to a depth below the sea floor of 30 metres 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, Reference [A.6.5-1], 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, vane shear, T-bar and/or pressure meter tests.
A.6.5.1.5.3 Geotechnical report

The geotechnical report should include borehole logs, in-situ test records (if appropriate) and documentation of all laboratory tests, together with interpreted soil design parameters. A competent geotechnical engineer should select design parameters suitable for spudcan foundation assessment. For the methods recommended in Section 9.3 and 9.4, the design parameters should include profiles of undrained shear strength and/or effective stress parameters, soil indices (plasticity, liquidity, grain size, etc.), relative density, unit weight and the over consolidation ratio (OCR).

Additional soil testing to provide shear moduli and cyclic/dynamic behaviour should be undertaken if more comprehensive analyses are to be applied or where the soil strength can deteriorate under cyclic loading.

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 location. 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 offered.

A.7 Actions

A.7.1 Scope

This clause presents applicable formulations and methods to calculate actions for site-specific assessments.

The wave and current actions are presented for quasi-static and dynamic analyses. 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 are also presented. Such analyses are applicable for calculation of inertial load sets or for the direct calculation of the structural responses including dynamic effects. The hydrodynamic formulations and coefficients are presented together with formulas 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.

Guidance on the determination of the functional actions is presented.

A.7.2 General

No guidance offered.

A.7.3 Metocean actions

A.7.3.1 General

A.7.3.1.1 Action combinations

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 of the following 100 year joint probability metocean data combinations:
   i) 100 year return period wave, the associated current and associated wind,
   ii) 100 year 1-minute wind, the associated wave and associated current,
   iii) 100 year current and the associated wave and associated wind.

A.7.3.1.2 Methods for the determination of actions

This clause 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 a single-degree-of-freedom (SDOF) method or a stochastic assessment of the wave actions to develop a dynamic amplification factor (DAF).

A more detailed stochastic time-domain analysis procedure implicitly includes inertial actions and can account for nonlinearities 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>
<thead>
<tr>
<th>TOPIC</th>
<th>Description</th>
<th>Deterministic</th>
<th>Stochastic DAF</th>
<th>Fully Integrated Stochastic</th>
</tr>
</thead>
<tbody>
<tr>
<td>Water depth</td>
<td>Define storm water depth considering LAT, tide and storm surge</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Wave</td>
<td>Specify wave height and range of associated wave periods.</td>
<td>A.6.4.2.2</td>
<td>A.6.4.2.3</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Define the return period significant wave height and corresponding spectral peak period.</td>
<td>A.6.4.2.5, A.6.4.2.7</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Calculate effective significant wave height as appropriate</td>
<td>A.6.4.2.6</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Specify wave spectrum, wave direction and wave spreading function.</td>
<td>A.6.4.2.5, A.6.4.2.8</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Wave theory</td>
<td>Calculate wave velocities and accelerations by superposition of wave components representing the wave spectrum and wave spreading functions.</td>
<td>A.7.3.3.3.2</td>
<td></td>
<td></td>
</tr>
<tr>
<td>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></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Apply a reduction factor to the wave kinematics</td>
<td>A.6.4.2.3</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Current</td>
<td>Define current velocity and profile.</td>
<td>A.6.4.3</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<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>A.7.3.3.4</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<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>A.7.3.3.3.2</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Scale environment</td>
<td>Apply partial factors to wind, wave and current to match factored deterministic actions</td>
<td>n/a</td>
<td></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, appurtenances</td>
<td>A.7.3.2.1, A.7.3.2.2, A.7.3.2.3, A.7.3.2.6</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
When a full integrated stochastic analysis is undertaken (10.3) partial factors are applied to the metocean parameters as described in A.10.5.3 and 8.8.1.3. Partial action factors are not applied when using stochastic dynamic analyses to determine a DAF for application in a deterministic analysis.

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 the clauses below. In all cases the following should be considered:

The drag properties of some chords differ for flow in the direction of the wave propagation (wave crest) and for flow back towards the source of the waves (wave trough). Often the combined drag properties of all the chords on a leg gives a total which is independent of the flow direction along a particular axis. When this is not the case it is recommended that the effect is included directly in the wave/current action model. Otherwise, where possible it is recommended that:

a) Regular wave deterministic calculations use a value appropriate to the flow direction under consideration, noting that the flow direction is that of the combined wave and current particle motion.

b) An average drag property is considered for random wave analyses which are solely used to determine dynamic effects for inclusion in a final regular wave deterministic calculation which is made on the basis of item a) above.

c) The drag property in the direction of wave propagation is used for random wave analyses from which the final results are obtained directly.
Lengths of members are normally taken as the node-to-node distance (see note below) of the members in order to account for small non-structural items (e.g. anodes, jetting lines of less than 4” nominal diameter). Large non-structural items such as raw water pipes and ladders are to be included in the model. Free standing conductor pipes and raw water towers are to be considered separately from the leg hydrodynamic model.

NOTE For the purpose of this calculation a node is defined as that 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 CD.

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 2.5Hs or where penetrations exceed 1/2 the spudcan height, the effect of the spudcan is normally insignificant.

On some jack-ups the lower section of the leg adjacent to the spudcan can be heavily reinforced for towage and 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 hydrodynamic coefficients formulation for jack-ups. Jack-up rigs are usually space frame structures with few parallel members in close proximity so that shielding and solidification effects are usually not important. 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 accounting for the combined shape with reference to relevant literature[A.7.3-1]. Model test data may be used if available. In such cases the effects of roughness, Keulegan-Carpenter and Reynolds number dependence should be considered.

The top of the spudcan is normally below or so close to the sea floor that it may be neglected in the hydrodynamic modelling. However, in case of special spudcan designs or very shallow penetrations, e.g. with the spudcan top more than 5% of the water depth above the sea floor, the hydrodynamic actions should be modelled with hydrodynamic coefficients applicable for large diameter members, see ISO TR 19905-2 Clauses 7.3.2.4 and 7.3.2.5.

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 CD = ΣCDiDi and CM = ΣCMiπDi2/4, 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) vn, un and vn are applied together with equivalent CD = ΣCDeDe and CM = ΣCMeMe, 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, CDei, times the equivalent diameter, Dei, for CDei of the bay can be chosen as:

\[ C_{De} D_e = D_e \sum C_{Dei} \]  

(A.7.3-1)
The equivalent value of the drag coefficient for each member, $C_{Dei}$, is determined from:

$$C_{Dei} = \left[ \sin^2 \beta_i + \cos^2 \beta_i \sin^2 \alpha_i \right]^{3/2} \frac{C_{Di}}{D_e s}$$  \hspace{1cm} (A.7.3-2)

where

- $C_{Di}$ = drag coefficient of an individual member (i) as defined in A.7.3.2.4
- $D_i$ = reference diameter of member 'i' (including marine growth as applicable) as defined in A.7.3.2.4.
- $D_e$ = equivalent diameter of leg, suggested as $\sqrt{\sum D_i^2 l_i / s}$
- $l_i$ = length of member 'i' node to node centre.
- $s$ = length of one bay, or part of bay considered.
- $\alpha_i$ = angle between flow direction and member axis projected onto a horizontal plane.
- $\beta_i$ = angle defining the member inclination from horizontal (see Figure A.7.3-1).

Note: $\Sigma$ indicates summation over all members in one leg bay.

The above expression for $C_{Dei}$ may be simplified for horizontal and vertical members as follows:

Vertical members (e.g. chords): 

$$C_{Dei} = C_{Di} \left( \frac{D_i}{D_e} \right)$$  \hspace{1cm} (A.7.3-3)

Horizontal members:

$$C_{Dei} = \sin^3 \alpha_i C_{Di} \frac{D_i l_i}{D_e s}$$  \hspace{1cm} (A.7.3-4)

**Figure A.7.3-1 — Flow angles appropriate to a lattice leg**

(after DNV Class Note 31.5, February 1992, Reference [A.7.3-2])

The equivalent value of the inertia coefficient, $C_{Me}$, and the equivalent area, $A_e$, representing the bay may be chosen as:

- $C_{Me}$ = equivalent inertia coefficient which may normally be taken as 2.0 when using $A_e$
- $A_e$ = equivalent area of leg per unit height = $\left( \sum A_i l_i / s \right)$
- $A_i$ = equivalent area of member or gusset = $\pi D_i^2 / 4$
D_i = reference diameter chosen as defined in A.7.3.2.4.

For a more accurate model the C_{Me} coefficient may be determined as:

\[ C_{Me} A_e = A_e \sum C_{Mei} \]  \hspace{1cm} (A.7.3-5)

where

\[ C_{Mei} = \left[ 1 + (\sin^2 \beta_i + \cos^2 \beta_i \sin^2 \alpha_i) (C_{Mi} - 1) \right] \frac{A_i}{A_e s} \]  \hspace{1cm} (A.7.3-6)

\[ C_{Mi} = \text{the inertia coefficient of an individual member, } C_{Mi} \text{ is defined in A.7.3.2.4 related to reference dimension } D_i. \]

NOTE 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.

A.7.3.2.4 Drag and inertia coefficients

Hydrodynamic coefficients for leg members are given in this Clause. 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 and current forces, the values of drag and inertia coefficients applicable to Morison's equation should be obtained from this Clause.

Recommended values for hydrodynamic coefficients for tubulars (<1.5m diameter) are given in Table A.7.3-3 based on the data discussed in the supporting ISO TR 19905-2 Clause 7.3.2.4.

<table>
<thead>
<tr>
<th>Surface condition</th>
<th>( C_{Di} ) for wave load analysis</th>
<th>( C_{Mi} ) for earthquake</th>
</tr>
</thead>
<tbody>
<tr>
<td>Smooth</td>
<td>0.65</td>
<td>2.0</td>
</tr>
<tr>
<td>Rough</td>
<td>1.00</td>
<td>1.8</td>
</tr>
</tbody>
</table>

The smooth values normally apply above MSL + 2m and the rough values below MSL + 2m, 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 MWL + 2m.

Hydrodynamic coefficients for large diameter members may be calculated according to ISO TR 19905-2 Clauses 7.3.2.4 and 7.3.2.5.

Actions due to gussets should be determined using a drag coefficient:

\[ C_{Di} = 2.0 \]

applied together with the projected area of the gusset visible in the flow direction, unless model test data shows 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, \( A = D_i l_i \) 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 formulas, as appropriate.

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 as:

$$
C_{Di} = \begin{cases} 
C_{Do} & ; \ 0^\circ < \theta \leq 20^\circ \\
C_{Do} + (C_{Di}W / D_i - C_{Do}) \sin^2 [(\theta - 20^\circ)\theta / 7] & ; \ 20^\circ < \theta \leq 90^\circ 
\end{cases} 
$$

(A.7.3-7)

where

- $t_m$ = marine growth thickness
- $\theta$ = angle in degrees, see Figure A.7.3-3
- $C_{Do}$ = the drag coefficient for a tubular with appropriate roughness, see Table A.7.3-3.
The drag coefficient for flow normal to the rack ($\theta = 90^\circ$), related to projected diameter, $W$. $C_{D1}$ is given by:

$$
C_{D1} = \begin{cases} 
1.8 & ; \quad W / D_i < 1.2 \\
1.4 + 1/3(W / D_i) & ; \quad 1.2 < W / D_i < 1.8 \\
2.0 & ; \quad 1.8 < W / D_i 
\end{cases}
$$

(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 for all heading angles and any roughness.

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 as:

$$
C_{Di} = C_{Dpr}(\theta) \frac{D_{pr}(\theta)}{D_i}
$$

(A.7.3-9)

where the drag coefficient related to the projected diameter, $C_{Dpr}$, is determined from:

$$
C_{Dpr}(\theta) = \begin{cases} 
1.70 & ; \quad \theta = 0^\circ \\
1.95 & ; \quad \theta = 90^\circ \\
1.40 & ; \quad \theta = 105^\circ \\
1.65 & ; \quad \theta = 180^\circ - \theta_o \\
1.00 & ; \quad \theta = 180^\circ 
\end{cases}
$$

(A.7.3-10)

Linear interpolation is to be applied for intermediate headings. The projected diameter, $D_{pr}(\theta)$, should be determined from:

$$
D_{pr}(\theta) = \begin{cases} 
D \cos \theta & ; \quad 0 < \theta < \theta_o \\
W \sin \theta + 0.5D \cos \theta & ; \quad \theta_o < \theta < 180^\circ - \theta_o \\
D \cos \theta & ; \quad 180^\circ - \theta_o < \theta < 180^\circ 
\end{cases}
$$

(A.7.3-11)

The angle $\theta_o$, where half the rackplate is hidden, $\theta_o = \tan^{-1}(D/(2W))$.

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[A.7.3-1] 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 location, 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 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 may normally be ignored.

The marine growth thickness may be ignored if anti-fouling, cleaning or other means are applied. The surface roughness is still to be taken into account, see A.7.3.2.4 or ISO TR 19905-2 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, 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 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 below. The wave and current velocities should be combined before they are used in the Morison equation.

The intrinsic wave period is based on a reference frame travelling with the speed and direction of the current, and is always used to calculate the wave kinematics. In general, when calculating the SDOF DAF the apparent wave period should be applied. For SDOF and stochastic calculations it may be acceptable to use the intrinsic wave period; e.g. when the wave period is greater than 10 seconds and $V_c < 1$ m/s the error is expected to be small.

Formulas for transformation between the intrinsic and apparent wave periods are given in ISO 19901-1 Clause A.8.3. The assessor should ensure that the correct procedure is used by the software in calculating wave particle kinematics and dynamics.

NOTE ISO 19901-1 uses terminology conflicting from that in API RP 2A, 21st Edition. 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" = RP 2A "apparent" and ISO 19901-1 "apparent" = RP 2A "actual".

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:

\[ \lambda > 5D_i \quad (A.7.3-12) \]

where

\[ \lambda = \text{wave length} \]

\[ D_i = \text{reference dimension of member (e.g. tubular diameter)} \]

The Morison equation specifies the action per unit length as the vector sum:

\[ \Delta F = \Delta F_{\text{drag}} + \Delta F_{\text{inertia}} = 0.5 \rho \left( C_D D \left| v_n \right| \right) + \rho C_M A \left( \cdot u_n - \alpha \rho C_A A^2 \right) \quad (A.7.3-13) \]

where the terms of the equation are described in the following.

To obtain the drag action, the appropriate drag coefficient \( C_D \) is to be chosen in combination with a reference diameter, including any increase for marine growth, as described in A.7.3.

The Morison drag action formulation is:

\[ \Delta F_{\text{drag}} = 0.5 \rho C_D D v_n \left| v_n \right| \quad (A.7.3-14) \]

where

\[ \Delta F_{\text{drag}} = \text{drag action (per unit length) normal to the axis of the member considered in the analysis and in the direction of } v_n. \]

\[ \rho = \text{mass density of water (normally 1025 kg/m}^3) \].

\[ C_D = \text{drag coefficient (} = C_{Di} \text{ or } C_{De} \text{ from A.7.3).} \]

\[ v_n = \text{relative fluid particle velocity resolved normal to the member axis.} \]

\[ D = \text{the reference dimension in a plane normal to the fluid velocity } v_n \]

\[ ( = D_i \text{ or } D_e \text{ from A.7.3).} \]

The relative fluid particle velocity, \( v_n \), may be taken as:

\[ v_n = u_n + V_{Cn} - \alpha \cdot r_n \quad (A.7.3-15) \]

where

\[ u_n + V_{Cn} = \text{the combined particle velocity found as the vectorial sum of the wave particle velocity and the current velocity, normal to the member axis.} \]

\[ \dot{r}_n = \text{the velocity of the considered member, normal to the member axis and in the direction of the combined particle velocity.} \]
\( \alpha = 0 \), if an absolute velocity is to be applied, i.e. neglecting the structural velocity.

\( \alpha = 1 \), if relative velocity is to be included. May only be used for stochastic/random wave action analyses if:

\[ u T_n / D_i \geq 20 \]

where \( u \) = particle velocity = \( V_C + \pi H_s / T \)

\( T_n \) = first natural period of surge or sway motion, and

\( D_i \) = the reference diameter of a chord.

**Note:** See also A.10.4.3 for relevant damping coefficients depending on \( \alpha \).

To obtain the inertia action, the appropriate inertia coefficient \( (C_M) \) is to 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. The Morison's inertia action formulation is:

\[
\Delta F_{\text{inertia}} = \rho C_M A \ddot{u}_n - \alpha \rho C_A A \ddot{r}_n \tag{A.7.3-16}
\]

where

\( \Delta F_{\text{inertia}} \) = inertia action (per unit length) normal to the member axis and in the direction of \( \ddot{u}_n \)

\( C_M \) = inertia coefficient

\( A \) = cross sectional area of member \((= A_i \text{ or } A_e \text{ from A.7.3.2.3})\)

\( \ddot{u}_n \) = fluid particle acceleration normal to member

\( C_A \) = added mass coefficient

\( = C_M - 1 \)

\( \ddot{r}_n \) = acceleration of the considered member, normal to the member axis and in the direction of the combined particle acceleration.

The last term in Equation (A.7.3-16) is the added mass force due to the member acceleration

\[
m_a \ddot{r}_n = \rho C_A A \ddot{r}_n
\]

where

\( m_a \) = added mass contribution (per unit length) for the member

In a dynamic response analysis the added mass \((m_a \text{ integrated over the member length})\) is normally transferred to the left hand side of the equation 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 Clause A.8.4, as shown in Figure A.7.3-1. 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 Clause A.8.4, it is recommended that the wave period is changed to comply with the breaking limit for the specified height.

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, A.6.4.2.6,
A.6.4.2.7, and A.6.4.2.8). It is recommended that the random sea state is 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 2-D 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 3-D 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 follows:

$$z' = \frac{z - \zeta}{1 + \zeta/d}$$

(A.7.3-17)

where

- $z'$ = the modified coordinate to be used in particle velocity formulation
- $z$ = the elevation at which the kinematics are required (coordinate measured vertically upward from the still water surface)
- $\zeta$ = the instantaneous water level (same axis system as $z$)
- $d$ = the still, or undisturbed water depth (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 Reference [A.7.3-3] and ISO TR 19905-2 Figure 7.3-23.

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 nonlinear (squared) transformation of wave kinematics, makes the hydrodynamic action excitation always nonlinear. 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. A number of procedures for doing this are presented in C.5.

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 under predict 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.7.3.4.

A.7.3.3.3.3 The effect of directionality and spreading on dynamic response

Both the magnitude of the loads 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:
Table A.7.3-4 — Recommendations for modelling of time domain stochastic waves

<table>
<thead>
<tr>
<th>Method</th>
<th>Recommendations</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Time Domain</strong></td>
<td>Generate random sea from at least 200 components and use divisions of generally equal energy. It is recommended that smaller energy divisions are 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[^A.7.3-4]. Unless indicated otherwise in the site specific information, the validity of wave surface simulation should be checked against the criteria below. For higher order waves criteria should be taken to assure that $H_s$, mean waves and maximum crests are within practical limits.</td>
</tr>
<tr>
<td></td>
<td>- correct mean wave elevation</td>
</tr>
<tr>
<td></td>
<td>- standard deviation = $(H_s/4) \pm 1%$</td>
</tr>
<tr>
<td></td>
<td>- $-0,03 &lt;$ skewness $&lt; 0,03$</td>
</tr>
<tr>
<td></td>
<td>- $2,9 &lt;$ kurtosis $&lt; 3,1$</td>
</tr>
<tr>
<td></td>
<td>- Maximum crest elevation = $(H_s/4) \sqrt{2 \ln(N)}$ $-5%$ to $+7,5%$</td>
</tr>
</tbody>
</table>

where $N$ is the number of cycles in the time series being qualified,

$N = \text{Duration} / T_z$

Integration time-step less than the smaller of:

$T_z/20$ or $T_n/20$

where

$T_z =$ the zero-upcrossing period of the wave spectrum

$T_n =$ 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 transients effects, discard at least the first 100 seconds (the ‘run-in’).

Ensure the simulation is of sufficient duration so that the method chosen results in a demonstrably stable Most Probable Maximum Extreme (MPME) response(s), see also A.10.5.3.4 and C.5.

Method 1: In a 2-D 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 Sec. 7.6.4 of Reference [A.7.3-4]:

a) Develop a set of 2-D BSTFs, one for the "principal" direction of interest, and the others offset from the principal direction.

[^A.7.3-4]: References to external documents or standards are provided for guidance and may not be part of the ISO/DIS 19905-1 standard.
b) For each offset direction, calculate a directionality contribution factor based in ISO 19901-1 A.8.7 or ISO TR 19905-2 6.4.2.7. 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 2-D BSTF (principal one plus the offset directions) multiplied by the corresponding directionality factors. Note that only the principal direction vectorial component of the offset direction BSTFs is used.

d) The BSTF for the chosen 2-D (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 2-D sea state should be chosen to obtain a match with the 3-D 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 should be carried out 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 would be with the weather approaching from approximately 15° or 45° off the bow, the dynamic amplification factors (DAF's) should be determined for one, or both, of these headings, see Fig. A.10.4-1. The DAF's (or more conservative DAF's) may be applied to the final deterministic analysis for all headings and hull weight cases with, when applicable, nonlinear fixity iterations according to A.10.4.4.

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 forces 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 on the flow field of the current, see Reference [A.7.3-5] and ISO TR 19905-2 7.3.3.4, as follows:

\[ V_C = V_f \left[ 1 + C_{De} D_e / (4D_F) \right]^{-1} \quad (A.7.3-18) \]

where

- \( V_C \) = the current velocity to be used in the hydrodynamic model, \( V_C \) should not be taken as less than 0,7\( V_f \).
- \( V_f \) = the far field (undisturbed) current.
- \( C_{De} \) = equivalent drag coefficient of the leg, as defined in A.7.3.2.3.
- \( D_e \) = equivalent diameter of the leg, as defined in A.7.3.2.3.
- \( D_F \) = face width of leg, outside dimensions, orthogonal to the flow direction.

A.7.3.4 Wind actions

A.7.3.4.1 Wind action

The wind force for each component (divided into blocks of not more than 15m vertical extent), \( F_{Wi} \), may be computed using the formula:

\[ F_{Wi} = P_i A_{Wi} \quad (A.7.3-19) \]

where
\[ P_i = \text{the pressure at the centre of the block.} \]

\[ A_{Wi} = \text{the projected area of the block considered.} \]

The pressure \( P_i \) should be computed using the formula:

\[ P_i = 0.5 \rho V_{zi}^2 C_s \quad (A.7.3-20) \]

where

\[ \rho = \text{density of air (to be taken as 1,2224 kg/m}^3\text{ unless an alternative value can be justified for the site).} \]

\[ V_{zi} = \text{the specified wind velocity at centre of each block, see A.6.4.6.2.} \]

\[ C_s = \text{shape coefficient, as given below.} \]

\[ \text{NOTE} \quad \text{The wind area of the hull and associated structures (excluding derrick and legs) can normally be taken as the} \]

\[ \text{projected area viewed from the direction under consideration.} \]

A.7.3.4.2 Shape coefficient

Using a building block of elements the shape coefficients in Table A.7.3-5 should be used.

<table>
<thead>
<tr>
<th>Type of member or Structure</th>
<th>Shape coefficient ( C_s )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Hull side, (flat side).</td>
<td>1.0 based on total projected area.</td>
</tr>
<tr>
<td>Hull and associated structures (excluding derrick and legs).</td>
<td>1.1 based on the total projected area (i.e. the area enclosed by the extreme contours of the structure)</td>
</tr>
<tr>
<td>Deckhouses, jack-frame structure, sub-structure, draw-works house, and other above-deck blocks.</td>
<td>1.1 based on the projected area</td>
</tr>
<tr>
<td>Leg sections projecting above jack-frame structure and below the hull.</td>
<td>( C_s = C_{De} ) as determined from A.7.3.2.3 using tubular ( C_{Di} = 0.5 ) ( A_{Wi} ) determined from ( D_{in} ) and section length.</td>
</tr>
<tr>
<td>Isolated tubulars (crane pedestals, etc.)</td>
<td>0.5</td>
</tr>
<tr>
<td>Isolated structural shapes (angles, channels, box, I-sections).</td>
<td>1.5 based on member projected area.</td>
</tr>
<tr>
<td>Derricks, crane booms, flare towers (open lattice sections only, not boxed- in sections).</td>
<td>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).</td>
</tr>
</tbody>
</table>

A.7.3.4.3 Wind tunnel data

Wind pressures and resulting actions for the hull and associated structures can 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, only the elevated storm mode need be assessed. 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 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 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:

a) The functional actions do not exceed the operations manual limits.

b) Procedures, equipment and instructions exist for performing the operation offshore.

c) 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 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 offered.

A.7.6 Dynamic effects

No guidance offered.

A.7.7 Earthquake

No guidance offered.

A.7.8 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 Structural modelling

A.8.1 Applicability

This clause describes methods for the development of an analytical model of an independent leg jack-up. Techniques for modelling the legs, hull, leg/hull connection, and leg/spudcan connection are discussed. The leg/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 analyst is familiar with the whole of this clause before commencing the modelling.

A.8.2 Overall considerations

A.8.2.1 General

No guidance 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 a Finite Element (FE) computer model.

A.8.2.3 Levels of FE modelling

While it can be considered 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 noted 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/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 (A.8.3.3), hull structure modelled using beam elements (A.8.4.3), leg to hull connections (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/hull 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 (A.8.3.4). The hull, leg/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/hull connections and the adequacy of the jacking and/or fixation systems. See Figure A.8.2-1.

![Combined equivalent/detailed leg and hull model](image)

**Figure A.8.2-1- Combined equivalent/detailed leg and hull model**

d) Detailed single leg and leg/hull connection model

The model consists of a 'detailed leg' or a portion of a 'detailed leg' (A.8.3.2), the leg/hull connection (A.8.5) and, when required, the spudcan (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.

<table>
<thead>
<tr>
<th>Model Type</th>
<th>Applicability</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>I</td>
</tr>
<tr>
<td>a) Fully detailed leg</td>
<td>Yes</td>
</tr>
<tr>
<td>b) Equivalent leg (stick model)</td>
<td>Yes</td>
</tr>
<tr>
<td>c) Combined equivalent / detailed leg and hull</td>
<td>Yes</td>
</tr>
<tr>
<td>d) Detailed single leg and leg/hull connection model</td>
<td>-</td>
</tr>
</tbody>
</table>

**Table A 8.2-1 — Applicability of the suggested models**

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 the applicable following subclause(s). Single column legs can be modelled with beam elements (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 Figure A.8.3-1 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 above. 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 referred to above can be accomplished as outlined below:

- From hand calculations using the formulae presented in Figure A.8.3-1. 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.

- From the application of unit load cases to a detailed leg model prepared in accordance with 8.3.2 and 8.3.5 and rigidly restrained, generally at the first point of lateral force transfer between the hull and leg, although it can be more convenient to use a different reference point e.g. chock level or neutral axis of the hull. 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, $A$, of the equivalent beam according to standard beam theory:
    \[
    \Delta = \frac{FL}{AE} \Rightarrow A = \frac{FL}{E\Delta}
    \]
    where
    \[
    \Delta = \text{axial deflection of cantilever at point of force application} \\
    F = \text{applied axial action} \\
    L = \text{cantilevered length (from the hull to seabed reaction point – see A.8.6.2)} \\
    E = \text{Young's modulus}
    \]
  - Pure moment applied either as a moment or a couple. This is used to derive the second moment of area ($I$) according to standard beam theory:
    \[
    \delta = \frac{ML^2}{2EI} \Rightarrow I = \frac{ML^2}{2E\delta} \quad \text{and} \quad \theta = \frac{ML}{EI} \Rightarrow I = \frac{ML}{E\theta}
    \]
ISO/DIS 19905-1

Figure A.8.3-1: Formulas for the determination of equivalent member properties (after DNV Class Note 31.5 1992 [A.8.3-1], corrected)

**Figure 5.1.a.**

<table>
<thead>
<tr>
<th>STRUCTURE</th>
<th>EQUIVALENT SHEAR AREA FOR TWO DIMENSIONAL LATTICE STRUCTURES</th>
<th>EQUIVALENT SHEAR AREA,</th>
</tr>
</thead>
<tbody>
<tr>
<td>A.</td>
<td></td>
<td>$A_{Qi} = \frac{(1+i)sh^2}{4A_D + h^2}x_i^2$</td>
</tr>
<tr>
<td>B.</td>
<td></td>
<td>$A_{Qi} = \frac{(1+i)sh^2}{2A_D + \frac{h^2}{2A_D} + \frac{x_i^2}{3A_D}}$</td>
</tr>
<tr>
<td>C.</td>
<td></td>
<td>$A_{Qi} = \frac{(1+i)sh^2}{4A_D + \frac{h^2}{2A_D} + \frac{x_i^2}{3A_D}}$</td>
</tr>
<tr>
<td>D.</td>
<td></td>
<td>$A_{Qi} = \frac{(1+i)sh^2}{2A_D + \frac{h^2}{2A_D} + \frac{x_i^2}{3A_D}}$</td>
</tr>
<tr>
<td>E.</td>
<td></td>
<td>$A_{Qi} = \frac{48(1+i)x_i^2}{6A_D + \frac{h^2}{2A_D} + \frac{x_i^2}{3A_D}}$</td>
</tr>
</tbody>
</table>

**Figure 5.1.b.**

<table>
<thead>
<tr>
<th>LEG.</th>
<th>EQUIVALENT SECTION PROPERTIES OF 3D LATTICE LEGS</th>
<th>EQUIVALENT PROPERTIES</th>
</tr>
</thead>
<tbody>
<tr>
<td>A.</td>
<td>$A = 3A_{CI}$</td>
<td>$A_{QY} = \frac{A_{Qz}}{2}A_{CI}$</td>
</tr>
<tr>
<td>B.</td>
<td>$I_y = I_z = \frac{1}{2}A_{CI}h^2$</td>
<td>$I_T = \frac{1}{4}A_{QI}h^2$</td>
</tr>
<tr>
<td>C.</td>
<td>$A = 4A_{CI}$</td>
<td>$A_{Qy} = A_{Qz} = 2A_{QI}$</td>
</tr>
<tr>
<td></td>
<td>$I_y = I_z = A_{CI}h^2$</td>
<td>$I_T = A_{QI}h^2$</td>
</tr>
</tbody>
</table>

3.) $A_{CI}$ MAY HAVE TAKEN AS THE CHORD AREA INCLUDING A CONTRIBUTION FROM THE RACK TEETH. (SEE NOTE TO SECTION 5.6.4.)

4.) THE FORMULAE WILL BE INACCURATE IF SIGNIFICANT OFFSETS EXIST BETWEEN BRACE WORK POINTS.

5.) FOR BRACING ACCORDING TO 5.1.a.c THE EQUIVALENT BEAM END ROTATIONS WILL NOT BE ACCURATE; THIS MAY BE IMPORTANT IF THIS MODELLING IS USED IN CONJUNCTION WITH ROTATIONAL FOUNDATION STIFFNESS.

Note: 1.) $A_{QI}$ IS DETERMINED FROM 5.1.a.

2.) THE STIFFNESS PROPERTIES ARE THE SAME FOR ALL DIRECTIONS UNLESS THE CHORDS HAVE DIFFERENT AREAS.
where

\[ \delta = \text{lateral deflection of cantilever at the point of moment application} \]

\[ M = \text{applied moment} \]

\[ \theta = \text{slope of cantilever at point of moment application} \]

It should be noted that the value of \( I \) resulting from the two equations can differ somewhat.

- Pure shear, \( P \), applied at the end of the leg which can be used to derive \( I \) according to standard beam theory:

\[ \theta = \frac{PL^2}{2EI} \implies I = \frac{PL^2}{2E\theta} \]

Using either this value of \( I \), or a value obtained from the pure moment case, the effective shear area, \( A_s \), can then be determined from:

\[ \delta = \frac{PL^3}{3EI} + \frac{PL}{A_s G} \implies A_s = \frac{7.8PLI}{3EI\delta - PL^2} \]

where

\[ G = \text{shear modulus} = \frac{E}{2.6} \text{ for Poisson’s ratio of 0.3.} \]

**A.8.3.4 Combination 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/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 offered

**A.8.4 Modelling the hull**

**A.8.4.1 General**

Recommended methods of modelling the hull structure are given in the following subclauses. 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/hull connection

A.8.5.1 General

The leg/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 are properly modelled in terms of stiffness, orientation and clearance. A simplified derivation of the equivalent leg/hull connection stiffness can be used for the equivalent stick-leg 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) Opposed or unopposed jacking pinions (see Figure A.8.5-2a),

c) Pin and yoke jacking system (see Figure A.8.5-2b).

d) Fixed or floating jacking system.

![Figure A.8.5-2a: Unopposed and opposed pinion arrangements](image-url)
Representative leg-hull connections are shown in Figure A.8.5-3a through Figure A.8.5-3d. The basic function of the leg-hull connection is to transfer forces between the leg and hull as follows:

1) horizontal shear is transferred by a set of horizontal forces in the lower guides and/or fixation system,

2) vertical force is transferred via a set of vertical forces in the support system,

3) 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.

![Figure A.8.5-3a: Representative leg-hull connection (fixed jacking with no fixation system)](image)
Figure A.8.5-3b: Representative leg-hull connection (floating jacking with no fixation system)

NOTE << When re-drawn, shock pads to be depicted more clearly with lower pad above main deck. All these figures will be updated to place the jacking/fixation at a more typical location. >>

Figure A.8.5-3c: Representative leg-hull connection (fixed jacking with fixation system)
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 axial force at the leg to hull connection, due to the environmental actions, are transferred largely by the fixation system because of its high rigidity. 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 jackups 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. Typical shear force and bending moment diagrams for this configuration are shown in Figure A.8.5-4.

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.8.5-5 and A.8.5-6.

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.8.5-7.

The leg/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).
Figure A.8.5-4: Leg shear force and bending moment - jack-ups with a fixation system
Figure A.8.5-5: Leg shear force and bending moment - jack-ups without a fixation system and having a fixed jacking system with opposed pinions
Figure A.8.5-6: Leg shear force and bending moment - jack-ups without a fixation system and having a fixed jacking system with unopposed pinions
Figure A.8.5-7: Leg shear force and bending moment - jack-ups without a fixation system and having a floating jacking system
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).
— the vertical pinion force component acting at an offset from the chord neutral axis.

The following techniques are recommended for modelling leg/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 are 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.8.5-8.

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 can simulate 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 noted that the shock pad stiffness characteristics are normally nonlinear and can change significantly over time.
Figure A.8.5-8: Correction of point supported guide model for finite guide length

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, the foundations of the fixation system and the lower guide structures at their connections to the hull.

A.8.5.7 Equivalent leg/hull stiffness

The determination of stiffnesses for the equivalent leg/hull connection model referred to in 8.5.7 can be accomplished by the following means:

- The application of unit load cases to a detailed leg model in combination with a detailed leg/hull connection model in accordance with 8.3.2 and 8.5. Unit load cases are applied, as described in A.8.3.3. In this instance 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/hull connection model as follows:
Axial 'unit' load case. This is used to determine the vertical leg/hull connection stiffness, $K_{vh}$, from the axial end displacements of the detailed leg model, $\Delta$, and the axial end displacements of the combined leg and leg/hull connection model, $\Delta_C$, under the action of the same 'unit' load case, $F$:

$$K_{vh} = \frac{F}{(\Delta_C - \Delta)} \quad \text{(A.8.5-1)}$$

Pure moment applied either as a moment or a couple. This is used to derive the rotational connection stiffness, $K_{rh}$, from either the end slopes, $\theta$ and $\theta_C$, or the end deflections, $\delta$ and $\delta_C$, of the two models under the action of the same end moment, $M$:

$$K_{rh} = M / (\theta_C - \theta) \quad \text{or} \quad K_{rh} = ML / (\delta_C - \delta) \quad \text{(A.8.5-2)}$$

Pure shear which can be used to determine the horizontal leg/hull connection stiffness, $K_{hh}$, in a similar manner, accounting for the rotational stiffness already derived. Normally the horizontal leg/hull connection stiffness can be assumed infinite.

If the model contains nonlinearities due to the inclusion of gap elements care should be taken to ensure that suitable levels of 'unit' load case are applied such that the derived stiffness is applicable to the analysis to be undertaken.

### 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 holes, 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 which 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 Clauses 9 and A.9. Upper or lower bound 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 conservatively 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 forces. This percentage would normally be 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 the simplest adequate spudcan-soil models should normally be used. These models should incorporate the maximum interpreted small strain stiffnesses and capacities (Clause 9). Soil stiffness degradation should not normally be included in an earthquake screening analysis. More detailed spudcan-soil interaction representations may be used.

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, 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.

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 & 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 nonlinear, 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 factored 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 (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 location, 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 location. 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 over predicted 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:

- the maximum extreme storm weight distributed according to A.8.8.2,
- guide clearances set to zero and
- the elevating system loads equalised 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 compliance 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 the increase in hull weight and the revised distribution i.e. no less than 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 loading is 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 nodal actions. Where nodal actions are used, a sufficient number should be applied to reflect the distributed nature of the loading, and it should be ensured that the correct total shear and overturning moment are achieved on each leg.

Similarly, the wind loading on the hull and associated structure can be applied as distributed or nodal actions. The application should also ensure 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 the correct total shear and overturning moment are achieved on each leg, and reflect the distributed nature of the loading.

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 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 needed 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 only be applied to structure above the lower guide. Application of actions to the leg below the lower guide would directly affect the internal leg forces and the foundation forces which are both important for site-specific analysis.

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 to be captured

- lateral displacement of the hull causes the functional actions to increase global OTM (global effects), and

- Euler amplification of local member forces increases member stresses (local effects).
The analyst should be cognizant 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 moment amplification factor B (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 $B = 1.0$ in the moment amplification equations (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 understands specifically which large displacement effects the software approximates (global and perhaps local) so that the correct value of B can be chosen for use in the moment amplification equations (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 A.8.

The correction term is: $-P_g/L$

where

- $P_g$ = The sum of the leg forces due to functional actions on legs at hull. This should include the weight of the legs above the hull.
- $L$ = 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 over-prediction 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 nonlinear foundation elements are used, the earthed spring approach over-predicts the horizontal foundation reactions and results in erroneous foundation responses. The over-prediction 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 and code checks should include appropriate terms to account for amplification of local moments (see A.12.4).
A.8.8.7 Conductor actions

An explicit model of the conductor is rarely warranted. The conductor tension can be modelled as a static force and the hydrodynamic action included in the jack-up’s global analysis model. Hydrodynamic action from the conductor should be modelled such that the effective line of action is 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 are often considered to 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

No guidance offered

A.9 Foundations

A.9.1 Applicability

No guidance offered.

A.9.2 General

No guidance offered.

A.9.3 Geotechnical analysis of independent leg foundations

A.9.3.1 Foundation modelling and assessment

A.9.3.1.1 General

This clause addresses the approaches to foundation modelling for:

- Response analysis
- 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 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). Larger jack-ups spudcans can be in excess of 20 m in diameter, with shapes varying with manufacturer and rig. Non-circular spudcans can be approximated by means of a disc with equivalent diameter. The bearing capacity formulae given in this section 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 bearing capacity.

NOTE Terms which are not defined in the text can be found in Clause A.4.9.
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 rig is given in Fig. A.10.4-2 and A.10.4-3.

There are certain aspects that are not covered in the checks described above, which should be considered; some of the more common examples are listed below:

— where the "long-term" (drained) soil bearing capacity is less than the "short term" (undrained) capacity e.g. in the case for overconsolidated clays or cohesive silts with significant sand seams.

— where a reduction of soil strength occurs due to cyclic loading. This can be of particular significance for silty soils and/or carbonate materials.

— where a potential for punch-through exists and an increase in spudcan penetration occurs e.g. due to cyclic loading,

— where horizontal seams of weak soil are located beneath the spudcan that can result in lateral bearing capacity/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 bearing capacity which can be used in the assessment checks.

A.9.3.1.3 Simple pinned foundation

This foundation treatment incorporates a simple preload and sliding check (subject to limitations), otherwise a check on bearing capacity in terms of vertical and horizontal (sliding) capacities should be performed.
Table A.9.3-1 — Approaches to foundation assessment

<table>
<thead>
<tr>
<th>Fixity treatment in response analysis</th>
<th>Foundation assessment</th>
<th>Acceptance category</th>
<th>Reference clause</th>
</tr>
</thead>
<tbody>
<tr>
<td>Pinned</td>
<td>Simple preload and sliding check (subject to limitations)</td>
<td>Level 1; Steps 1a &amp; b</td>
<td>A.9.3.6.2</td>
</tr>
<tr>
<td></td>
<td>Bearing and sliding checks using vertical-horizontal capacity envelope</td>
<td>Level 2; Step 2a</td>
<td>A.9.3.6.4</td>
</tr>
<tr>
<td></td>
<td>Displacement check (requires that sliding meets one of the two preceding checks and with input from the vertical-horizontal capacity envelope and load-penetration curve)</td>
<td>Level 3; Step 3a</td>
<td>A.9.3.6.6</td>
</tr>
<tr>
<td>Fixity:</td>
<td>Fixity:</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Simple interaction surface (secant model)</td>
<td>Fixity:</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Bearing and sliding checks using vertical-horizontal capacity envelope</td>
<td>Level 2; Step 2b</td>
<td>A.9.3.6.5</td>
</tr>
<tr>
<td></td>
<td>Displacement check (requires that sliding meets one of the two preceding checks and with input from the vertical-horizontal capacity envelope and load-penetration curve)</td>
<td>Level 3; Step 3a</td>
<td>A.9.3.6.6</td>
</tr>
<tr>
<td></td>
<td>Full interaction surface (yield interaction model)</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Foundation checks are implicit in the nonlinear model</td>
<td>Level 2; Step 2c OR Level 3; Step 3b</td>
<td>A.9.3.6.5 A.9.3.6.6</td>
</tr>
<tr>
<td></td>
<td>Continuum</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Foundation checks are implicit in the nonlinear model</td>
<td>Level 3; Step 3b</td>
<td>A.9.3.6.6</td>
</tr>
</tbody>
</table>

**A.9.3.1.4 Linear vertical, linear horizontal and secant rotational stiffness**

This foundation fixity treatment incorporates a check on bearing capacity in terms of vertical and horizontal (sliding) capacities. The amount of rotational fixity is not directly involved in a checking equation. However, the moment, bearing and sliding interaction is implicitly checked through the use of the yield function. Bearing and sliding should still be checked explicitly through the procedures described in A.9.3.6.

**A.9.3.1.5 Nonlinear vertical, horizontal and rotational stiffness**

The moment, bearing and sliding interaction is implicitly checked through the use of the yield function as described in A.9.3.3. No other checks are required.
A.9.3.1.6 Nonlinear 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 nonlinear 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:

1) Model the spudcan.

2) Compute the gross ultimate bearing capacity, \( Q_V \), of an open hole for various depths of the bearing area below sea floor using closed form bearing capacity solutions.

3) Convert the gross ultimate vertical bearing capacity at each depth to the available structural spudcan reaction, \( V_L \), by deducting, when appropriate: the effect of submerged weight of the backfill, \( W_{BF} \), and adding the effect of spudcan soil buoyancy, \( B_S \), determined using the submerged soil weight.

\[
V_L = Q_V \\
V_L = Q_V - W_{BF} + B_S
\]

when there is no backfill

when there is backfill

(A.9.3-1.a)

(A.9.3-1.b)

4) 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. 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 that associated with 1.5 times the preload reaction.

5) Enter the curve of available structural spudcan reaction versus spudcan penetration curve with the maximum preload reaction at the spudcans and read off the predicted spudcan penetration.

NOTE << To improve clarity, a figure is to be added depicting backflow, infill, etc. >>

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 depth, \( D \), 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, may need 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 (\( \beta \), Figure 9.3-2 and Annex E) for the amount of spudcan penetrated should be evaluated. The equivalent cone should be taken to enclose the same volume as that of the spudcan below sea floor. The volume of the equivalent cone is equal to the volume of the penetrated
portion of the underside of the spudcan (up to the largest cross sectional area in plan) and the planar area in soil contact is consistent (as in Figure A.9.3-3). When a spudcan has embedded past its largest cross sectional area the equivalent conical angle is constant. Figure A.9.3-3 shows the definition of the equivalent bearing area for a variety of penetration conditions.

Figure A.9.3-2 — Calculating an equivalent conical spudcan for various embedments (after Martin, 1994)

NOTE << Figure to be updated to use diameter B not radius R & Beff (or similar) not Requiv. Penetration depth D to be added. Bibliographic reference for Martin, 1994 needed >>

NOTE << Left side of Figure to be labelled 'partially embedded' right side 'fully embedded'. Upper part of the figure to be identified as the real case and the lower part of the figure as the model. Insert "D" in the lower right figure. Move to below 9.3.2.2. Change plan of real case to one with notches. Also add tip penetration depth. Change Q to F. >>
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 condition 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.

The 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 noted that a man-made 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 the following clauses address the failure mechanisms shown in Figure A.9.3-4.

---

NOTE << When re-drawn: B) to be revised with different slopes in the different layers; D) inner shear arrows to be removed; E) the symbol for lower sand to be deleted. Updated drafts expected from Leung. >>
Figure A.9.3-4 — Spudcan bearing failure mechanisms

A.9.3.2.1.4 Backfill

Soil backfill on top of the spudcan can result from backflow or infill. Regardless of the mechanism, this soil:

i) increases penetration if it occurs during preloading,

ii) reduces capacity available to support downward structural loads at the spudcan if it occurs after preloading and

iii) 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.

The submerged weight of backfill \( W_{BF,0} \) during preloading loads the top of the spudcan and results in additional penetration.

The weight of the preload ballast per leg that is dumped after preloading is equal to the maximum available storm component of the vertical bearing reaction, from overturning, that can be supported by the foundation. The submerged weight of any subsequent backfill \( W_{BF,A} \) reduces that available storm bearing reaction; i.e. there is additional load on top of the spudcan that needs to be reacted and therefore reduces the amount of storm bearing load that the foundation can support. Conversely, subsequent backfill increases the available uplift capacity of the windward legs.

The minimum value of the backfill weight, due to backflow during preloading, depends on the limiting depth of cavity, \( H_{cav} \), that remains open above the spudcan during penetration:

\[
W_{BF,omin} = \gamma A (D - H_{cav})
\]  

Where \( H_{cav} \) can be taken from Figure A.9.3-5. When \( H_{cav} \geq D \) (no backflow) then \( W_{BF,omin} = 0 \).

The penetration resistance (normalised by the local shear strength, \( s_u \)) offered by a localised backflow mechanism becomes independent of depth for penetrations exceeding \( B \), whereas that offered by a conventional bearing capacity failure mechanism increases with depth, even though conventionally a limiting bearing capacity factor of 9 is generally adopted (see Table 9.3-1). The transition between these two penetration mechanisms governs the onset of backflow and hence the limiting cavity depth, \( H_{cav} \). \( H_{cav} \) can be estimated either by comparing the resistance offered by each failure mechanism, or using the curve shown in Figure A.9.3-5, which applies to clay with uniform or increasing strength with depth (Hossain, Reference [A.9.3-1]).

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.9.4.4), moment restraint and 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).
NOTE << When re-drawn, depth axis to be extended below the B dimension line. >>

Figure A.9.3-4 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 gross ultimate bearing capacity, $Q_{Vo}$, required to support the spudcan is equal to the preload reaction, $V_{Lo}$, plus the effective submerged weight of any backfill onto the spudcan, less the effective submerged weight of the soil displaced by the spudcan:

$$Q_{Vo} = V_{Lo} + W_{BF,o} - B_S$$

(A.9.3-3)

Where

- $W_{BF,o}$ is the submerged weight of the overburden on top of the spudcan from backfill during preloading
- $B_S$ is the submerged weight of soil displaced by the spudcan, $\gamma'V$.

The initial vertical bearing capacity, $Q_{Vo}$, is established by preload operations and related to $V_{Lo}$. However, in some cases, subsequent actions may cause further penetration and a corresponding increase in $Q_V$, as would be consistent with the load-penetration equations given in A.9.3.2.3 and A.9.3.2.5.

### 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 by:

$$Q_V = (s_u N_c s_d + p_o) \pi B^2 / 4$$

(A.9.3-4)

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 Reference [A.9.3-1]. However, these factors are significantly affected by the gradient of shear strength with depth (Young et al, Reference [A.9.3-2]; Houlsby and Martin, Reference [A.9.3-3]).

Theoretical solutions for circular conical foundations on clays of uniform and increasing strength with depth have been provided by Houlsby and Martin [A.9.3-3], as presented in Annex E. The solutions give a theoretical lower bound to the soil resistance, and should therefore provide an upper bound prediction of penetration.
The bearing capacity factors for rough spudcans are given in Table 9.3-2 and are valid for the following parameter ranges (see Figures A.9.3-2, A.9.3-4 and A.9.3-5):

- cone angles $\beta$ between 60° and a flat plate of 180°;
- embedment depths $D$ between zero and 2.5 diameters;
- values of shear strength gradient $\rho \Delta s_{um}$ between 0 and 5 where $\rho$ is the rate of increase in shear strength with depth, from a value of $s_{um}$ at the sea floor).

The tables in Annex E provide a theoretical lower bound to the bearing factor $N_{c,s_{c},d_{c}}$ to apply to the shear strength at the spudcan base level, $s_{um}$, for the full range of the above parameters. Alternatively, Houlsby and Martin[A.9.3-3] indicate that using the shear strength, $s_{um}$, at a depth of 0.09 $B$ below the spudcan base level together with the bearing factors given in Table 9.3-2 for a foundation on uniform strength clay provides answers which are within ±12% of the theoretical lower bound solutions.

Alternatively, field experience in the Gulf of Mexico[A.9.3-4] has indicated that for typical Gulf of Mexico shear strength gradients and spudcan dimensions, spudcan penetrations in clay are well predicted by selecting $s_{um}$ as the average over a depth of $B/2$ below the widest cross section in combination with the use of Skempton[A.9.3-1] bearing capacity and depth factors.

For clay layers with distinct strength differences methods for layered soils should be used (A.9.3.2.8).

### Table A.9.3-2 Bearing capacity factors for rough circular plate[A.9.3-3]

<table>
<thead>
<tr>
<th>Embedment ratio, $D/B$</th>
<th>Bearing factor, $N_{c,s_{c},d_{c}}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>6,0</td>
</tr>
<tr>
<td>0,1</td>
<td>6,3</td>
</tr>
<tr>
<td>0,25</td>
<td>6,6</td>
</tr>
<tr>
<td>0,5</td>
<td>7,1</td>
</tr>
<tr>
<td>1,0</td>
<td>7,7</td>
</tr>
<tr>
<td>≥ 2,5</td>
<td>9,0</td>
</tr>
</tbody>
</table>

#### 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) are performed to estimate the range of penetrations. Cyclic loading may significantly affect the bearing capacity of silts.

Penetration in soils with partial drainage can be assessed using the approaches described by Finnie & Randolph, Reference [A.9.3-4] and Erbrich [A.9.3-5].

#### 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:

$$Q_v = \gamma N_d \pi B^2/8 + p_o N_q \pi B^2/4$$  \hspace{1cm} (A.9.3-5)

where $B$ is the maximum spudcan diameter in contact with the soil, $\gamma$ the effective unit weight of the soil and $N_d$ 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, $p_0'$, 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 (Martin, Reference [A.9.3-6]) 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 (e.g. Table A.9.3-3 shows that a 1° change in $\phi'$ 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$^{[A.9.3-6]}$)

<table>
<thead>
<tr>
<th>Friction angle $\phi'$ (degrees)</th>
<th>Bearing factor, $N_γ$</th>
<th>Bearing factor, $N_q$</th>
</tr>
</thead>
<tbody>
<tr>
<td>20</td>
<td>2.4</td>
<td>9.6</td>
</tr>
<tr>
<td>21</td>
<td>2.9</td>
<td>10.9</td>
</tr>
<tr>
<td>22</td>
<td>3.5</td>
<td>12.4</td>
</tr>
<tr>
<td>23</td>
<td>4.2</td>
<td>14.1</td>
</tr>
<tr>
<td>24</td>
<td>5.1</td>
<td>16.1</td>
</tr>
<tr>
<td>25</td>
<td>6.1</td>
<td>18.4</td>
</tr>
<tr>
<td>26</td>
<td>7.3</td>
<td>21.1</td>
</tr>
<tr>
<td>27</td>
<td>8.8</td>
<td>24.2</td>
</tr>
<tr>
<td>28</td>
<td>10.6</td>
<td>27.9</td>
</tr>
<tr>
<td>29</td>
<td>12.8</td>
<td>32.2</td>
</tr>
<tr>
<td>30</td>
<td>15.5</td>
<td>37.2</td>
</tr>
<tr>
<td>31</td>
<td>18.8</td>
<td>43.2</td>
</tr>
<tr>
<td>32</td>
<td>22.9</td>
<td>50.3</td>
</tr>
<tr>
<td>33</td>
<td>27.9</td>
<td>58.7</td>
</tr>
<tr>
<td>34</td>
<td>34.1</td>
<td>68.7</td>
</tr>
<tr>
<td>35</td>
<td>41.9</td>
<td>80.8</td>
</tr>
<tr>
<td>36</td>
<td>51.6</td>
<td>95.4</td>
</tr>
<tr>
<td>37</td>
<td>63.7</td>
<td>113.0</td>
</tr>
<tr>
<td>38</td>
<td>79.1</td>
<td>134.4</td>
</tr>
<tr>
<td>39</td>
<td>98.7</td>
<td>160.5</td>
</tr>
<tr>
<td>40</td>
<td>123.7</td>
<td>192.7</td>
</tr>
</tbody>
</table>

**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, Reference [A.9.3-7]; Le Tirant and Nauroy, Reference [A.9.3-8]; Finnie and Randolph, Reference [A.9.3-9]). 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, Reference [A.9.3-10]). 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, Reference [A.9.3-11]; Pan, Reference [A.9.3-12]; Pan et al., Reference [A.9.3-13]; Byrne and Houlsby, Reference [A.9.3-14]) and theoretical studies (Yeung, Reference [A.9.3-15]) studies 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, Reference [A.9.3-7]; Houlsby et al., Reference [A.9.3-16]; Sharp and van Seters, Reference [A.9.3-17]; Randolph and Erbrich, Reference [A.9.3-18]). 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 [A.9.3-18] 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. The conventional method is to use the plasticity based formulation for bearing capacity of shallow foundations in sand. However, friction angles used in the formulae should be considerably smaller than laboratory values to account (in an artificial manner) for the high soil compressibility; typically a reduction of 3 to 7 degrees can be considered.

NOTE << The range 3 to 7 degrees to be confirmed by the Technical Panel. >>

Other predictive methods for circular spudcans on both cemented and uncemented calcareous sands have been published, including Islam, Reference [A.9.3-19]; Islam et al., Reference [A.9.3-20]; Houlsby et al., Reference [A.9.3-16]; Randolph et al., Reference [A.9.3-21]; Finnie and Randolph, Reference [A.9.3-9]; and Yamamoto et al. Reference [A.9.3-22] and [A.9.3-23]. 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.[A.9.3-20], [A.9.3-21] 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:

1) General shear.
2) Squeezing.
3) 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 $\phi$) can be determined below the spudcan. The spudcan penetration versus foundation capacity relationship is then generated using criteria from A.9.3.2.3 through A.9.3.2.7.
Criteria for the other two failure mechanisms (squeezing and punch-through) are given below. The latter condition 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 bearing capacity which results in rapid leg penetration.

Backflow and infill should be considered.

**A.9.3.2.6.2 Squeezing of clay**

On a soft clay subject to squeezing overlaying a significantly stronger layer (see Figure A.9.3-6), the gross ultimate vertical bearing capacity of a spudcan can be analyzed by methods given by Brown and Meyerhof, Reference [A.9.3-24] and by Vesic [A.9.3-25] in combination with the Skempton bearing capacity and depth factors[A.9.3-1].

\[
Q_v = A \left( a_s + \frac{b_s B}{T} + \frac{12 D}{B} \right) s_u + p_s^A \geq A \left( N_c s_u d_c s_u + p_s^A \right) \tag{A.9.3-6}
\]

where the following squeezing factors are recommended:

- \( a_s = 5.00 \)
- \( b_s = 0.33 \) for spudcans with \( B \leq 15 \) m and \( 0.25 \) for spudcans with \( B > 15 \) m.

and \( s_u \) refers to the undrained shear strength of the soft clay layer.

**Figure A.9.3-6: Spudcan bearing capacity analysis - squeezing clay layer**

It is noted that the lower bound foundation capacity is given by general failure in the clay layer (right hand side of equation), and that squeezing occurs when \( B \geq T/b \). The upper bound capacity (for \( T<<B \)) is determined by the ultimate bearing capacity of the underlying strong soil layer.

**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[A.9.3-24] (see Figure a.9.3-7):

\[
Q_v = A \left( 3 \frac{H}{B} s_{u,t} + N_c s_c s_{u,w} \right) \leq A N_c s_c s_{u,t} \tag{A.9.3-7}
\]

The equation above applies to clay layers of uniform undrained shear strengths.
A.9.3.6.4 Punch-through: Sand overlying clay

The gross ultimate vertical capacity of a spudcan on a sand layer overlying a weak clay layer can be computed using a load spread model (see Figure A.9.3-8). In this model the bearing capacity of the foundation is assumed to be equal to the bearing capacity of the foundation projected onto the lower layer for a given load spread. Typical load spread factors (n) for sand overlying clay are in the order of 3 to 5.

Figure A.9.3-7 — Spudcan bearing capacity analysis - two clay layers

NOTE << When re-drawn, change “strong” to “stronger” and “weak” to “weaker” and cu to su. Change Q to F, delete V >>

NOTE << When re-drawn, remove “Dense” and “weak” from the figure. Change Q_v at top to F_v in equation change Q_u,b to Q_u,b. Delete V, define W as weight of sand between base of footing and weak clay >>
Alternatively, the bearing capacity may be calculated using the following equations derived from Hanna and Meyerhof\cite{A.9.3-26}:

\[ Q_V = Q_{u,b} - AH\gamma' + 2AH(\gamma' + 2p_0)K_s\tan\phi / B \]  

(A.9.3-9)

where

\( Q_{u,b} \) is determined according to Section A.9.3.2.3, assuming 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\cite{A.9.3-26}.

A new approach based on a centrifuge study has been proposed by Teh\cite{A.9.3-27}. The load-penetration curve typical of the punch-through condition is represented by a simplified profile consisting of three characteristic bearing resistances, namely surface bearing resistance, \( q_0 \) (at \( d = 0 \)), maximum bearing resistance, \( q_{\text{max}} \) (at \( d = d_{\text{crit}} \)), and bearing resistance in the underlying clay (for \( d \geq H \)). A brief description of the approach is provided in Annex E.
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 foundation bearing capacity for 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.9.3-10) 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.9.3-9). Analysis for the top layer can incorporate load spread effects.

Use 2 layer bearing capacity procedures for both analyses

- Analysis 2: Layer 1 over (Layers 2 and 3)
- Analysis 1: Layers 2 over layer 3

NOTE << When re-drawn, switch the left and right sides of the diagram and footnote. >>

Figure A.9.3-10 — 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 which, with the vertical capacity, are the principal coordinates of the yield interaction surface.

The limiting combinations of the spudcan moment, vertical and horizontal reactions are defined by the yield interaction surface. Inside the yield surface the foundation behaviour is considered to be elastic for small strains but 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.

The following clauses are applicable to traditional spudcan designs. Information on the foundation behaviour of spudcans fitted with skirts can be found in References [A.9.3-28] to [A.9.3-32].

The modelling approach to the interaction of vertical, horizontal and rotational forces was initially developed for shallow foundations based on a plasticity relationship, see References [A.9.3-33] to [A.9.3-37]. 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 parabolic.

In clay, a deeply embedded spudcan can achieve a greater moment capacity than a spudcan with a shallow penetration (see References [A.9.3-38] to [A.9.3-40]). In addition, the shape of the yield surface changes from parabolic to become progressively more elliptical with increasing penetration (This was first shown experimentally by Martin and Houlsby[A.9.3-41], further substantiated via numerical analysis by Martin and Houlsby[A.9.3-42] and confirmed via finite element analysis by Templeton et al.[A.9.3-39]). This effect can be taken into account by interpolating between the parabolic shape of the shallow embedment yield surface (obtained by setting \( a = 0 \) in Equation A.9.3-10) and the elliptical shape for deep embeddings (\( D > 2.5B \)) using the depth parameter, \( a \). Accomplishment of the necessary interpolation via a single-parameter linear variation of the coefficients was shown to be sufficiently accurate by Templeton, et al.[A.9.3-39].

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 no existing data 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 equation, the vertical bearing capacity, \( Q_V \), is initially established by preload operations and related to \( V_{1o} \) as specified by the equation given in A.9.3.2.2.5. However, in some cases, subsequent environmental actions may cause further penetration and a corresponding increase in \( Q_V \), as would be consistent with the load-penetration equations given in A.9.3.2.3 through A.9.3.2.8. 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 equations, together with values of any additional penetration. In either case, care should be taken to include all contributions from P-delta effects associated with leaning due to the additional penetration. Consideration should be given to the possibility of excess penetration, rapid penetration and/or punch-through.

In the expression for the yield interaction surface, if a force combination \((F_V, F_H, F_M)\) satisfies the equality then \((F_V, F_H, F_M) = (Q_V, Q_H, Q_M)\). The force combination \((F_V, F_H, F_M)\) lies outside the yield surface if the left-hand side is greater than zero. Conversely, the force combination lies inside the yield surface if the left-hand side is less than zero.
A.9.3.3.2 Ultimate vertical / horizontal / rotational capacity interaction function for spudcans in sand and clay

The following general equation can be used for fully or partially penetrated spudcans:

\[
\left( \frac{F_{H}}{Q_{H}} \right)^2 + \left( \frac{F_{M}}{Q_{M}} \right)^2 - 16(1-a) \left( \frac{F_{V}}{Q_{V}} \right)^2 \geq 0 \tag{A.9.3-10}
\]

Where

- \( Q_{V} \) = the gross vertical bearing capacity of the soil beneath the spudcan. In the absence of additional penetration = \( Q_{V0} \) the capacity achieved during preloading, as defined in A.9.3.2.5,
ISO/DIS 19905-1

\[ F_V = \text{gross bearing reaction under storm conditions} \]

\[ \begin{align*}
V_{st} &= \text{no backfill} \\
V_{st} + W_{BF.o} + W_{BF.A} - B_S &= \text{with backfill}
\end{align*} \]

\[ V_{st} = \text{the structural vertical reaction beneath the spudcan due to the factored actions as determined from the procedures given in Clause 10, including leg weight and water buoyancy but excluding submerged weight of backfill (} W_{BF.o} + W_{BF.A} \text{) and spudcan soil buoyancy (} B_S \text{).} \]

NOTE The soil buoyancy of the spudcan in the formulation above is a simplification based on the spudcan either being completely buried or not buried at all. A more precise formulation can be used to account for partial embedment.

and \( Q_H \) and \( Q_M \) are defined as follows:

For Clay:

\[ Q_H = C_H Q_{Vnet} \quad \text{(see Note 1)} \quad \text{(A.9.3-11)} \]

\[ Q_M = (0,1 + 0,05a(1+b/2)) Q_{Vnet} B \quad \text{(A.9.3-12)} \]

with

\[ Q_{Vnet} = (s_u N_c s_c d_c) \pi B^2 / 4 = Q_V - \pi B^2 p_c / 4 \quad \text{(see Note 2)} \quad \text{(A.9.3-13)} \]

\[ C_H = C_{H\text{shallow}} + (C_{H\text{deep}} - C_{H\text{shallow}}) D / B \quad \text{for } D < B \quad \text{(A.9.3-14a)} \]

\[ C_H = C_{H\text{deep}} \quad \text{for } D \geq B \quad \text{(see Note 3)} \quad \text{(A.9.3-14b)} \]

\[ C_{H\text{shallow}} = (s_{uo} A + (s_{uo} + s_{ul}) A_s) / Q_{Vnet} \quad \text{(A.9.3-15)} \]

\[ C_{H\text{deep}} = 1,3 (A_s / A) \quad \text{(A.9.3-16)} \]

\[ a = D / 2,5B \quad \text{for } D < 2,5B \quad \text{(see Note 4)} \]

\[ a = 1,0 \quad \text{for } D \geq 2,5B \]

\[ b = (D_b s_t) / (D s_u) \quad \text{(see Note 3)} \]

\[ A = \text{spudcan effective bearing area based on cross-section taken at uppermost part of bearing area in contact with soil (see Figure A.9.3-3).} \]

\[ A_s = \text{spudcan laterally projected embedded area.} \]

\[ D_b = \text{depth of backflow (see A.9.3.2.1.4). Infill should not be considered.} \]

\[ s_t = \text{backflow soil strength} \]

\[ s_u = \text{undisturbed soil strength} \]

For sand:

\[ Q_M = 0,12 Q_{Vnet} \quad \text{(see Note 2)} \quad \text{(A.9.3-17)} \]

\[ Q_H = 0,075 Q_{Vnet} B \]

The horizontal and vertical shape of the yield surface are defined by the values of 0,12\( Q_{Vnet} \) and 0,075\( B Q_{Vnet} \) respectively. These are appropriate values that summarize experimental evidence that includes Tan,
The equation above only applies when:

- The application of these parameters, which are calibrated to shallow penetrations, is likely to be conservative for the deep penetration case.

At zero vertical load a shallow sand foundation has no horizontal or moment capacity because it is cohesionless and conforms to the yield interaction equation 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 \( F_V < 0.5 \frac{Q_V}{V} \). 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 that 0,0 and the yield surface may be enlarged; the simplified expansion given in A.9.3.3 should not be used.

**NOTE 1** The moment capacities, as well as the horizontal capacity in sand, are calculated as fractions of the net vertical soil capacity. This is very useful because in most jack-up applications, the vertical soil capacity established by the preload is known with greater accuracy than the soil strength. In some applications for clay, it is more convenient to calculate the horizontal capacity as a fraction of the vertical. For the shallow embedment case, a conservative value for this fraction 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 6 times the shear strength, so that: \( Q_H = 0.16 \frac{Q_{Vnet}}{V} \). This value can be used as an alternate, conservative, horizontal capacity expression for shallow embedment in clay.

**NOTE 2** \( Q_{Vnet} \) can normally be taken as \( V_{LS} \) without significant error, unless there is a significant cavity above the spudcan.

**NOTE 3** Both \( D \) (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 4** The depth interpolation parameter, \( a \), is given as a function of the embedment, \( D \), which is measured as the depth below mudline of the lowest point of the spudcan’s maximum width. Technically, \( D = 0 \) 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 \( a = 1 \) can be used.

In many cases, simpler forms of the yield interaction equation can be used. Results from finite element analysis (see Templeton et al., Reference [A.9.3-39] or Templeton, Reference [A.9.3-40]) indicate that insignificant error would be 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 equation reduces to the parabolic form:

\[
\left[ \frac{F_H}{Q_H} \right]^2 + \left[ \frac{F_M}{Q_M} \right]^2 - 16 \left[ \frac{F_V}{Q_V} \right]^2 \left( 1 - \frac{F_V}{Q_V} \right)^2 = 0
\]

(A.9.3-18)

In the case of \( a = 1 \), the yield interaction equation reduces to the fully ellipsoidal form:

\[
\left[ \frac{F_H}{Q_H} \right]^2 + \left[ \frac{F_M}{Q_M} \right]^2 - 4 \left[ \frac{F_V}{Q_V} \right] \left( 1 - \frac{F_V}{Q_V} \right) = 0
\]

(A.9.3-19)

The expression for the yield surface can be rewritten to give the maximum spudcan moment as a function of the horizontal and vertical forces. For a given vertical and horizontal force combination which lies inside the yield surface given above, the spudcan moment cannot exceed the maximum available moment capacity \( Q_M \):

\[
Q_M = Q_M \left[ 16(1-a) \left[ \frac{F_V}{Q_V} \right]^2 \left( 1 - \frac{F_V}{Q_V} \right)^2 - \left[ \frac{F_H}{Q_H} \right]^2 \right] + 4a \left[ \frac{F_V}{Q_V} \right] \left( 1 - \frac{F_V}{Q_V} \right) \right]^{0.5}
\]

(A.9.3-20)

The equation above only applies when:
0.0 < \( F_V < Q_V \)

and

\[
0.0 < 16(1 - \alpha) \left[ \frac{F_V}{Q_V} \right]^2 \left[ 1 - \frac{F_V}{Q_V} \right] - \left[ \frac{F_M}{f_2Q_M} \right]^2 + 4\alpha \left[ \frac{F_V}{Q_V} \right] \left[ 1 - \frac{F_V}{Q_V} \right] \]
\]  

(A.9.3-21)

### A.9.3.3.3 Spudcans in clay with \( F_V < 0.5 \) \( Q_V \)

As discussed in A.9.3.3.2.2, the yield surface at \( 0.0 < F_V/Q_V < 0.5 \) (typically applicable to windward legs) can be expressed as:

\[
\left[ \frac{F_H}{f_1Q_H} \right]^2 + \left[ \frac{F_M}{f_2Q_M} \right]^2 - 1.0 = 0
\]

(A.9.3-22)

where

\[
f_1 = \alpha + 2 \left( 1 - \alpha \right) \left[ \frac{F_V}{Q_V} \right]
\]

(A.9.3-23)

\[
f_2 = f_1 \text{ where suction (i.e. uplift resistance) is available,}
\]

(A.9.3-24a)

\[
f_2 = 4 \left[ \frac{F_V}{Q_V} \right] \left[ 1 - \frac{F_V}{Q_V} \right] \text{ where suction cannot be relied upon}
\]

(A.9.3-24b)

\[
\alpha = 1.0 \text{ for soft clays}
\]

\[
= 0.5 \text{ for stiff clays}
\]

\( \alpha \) accounts for the degree of adhesion. The assessor should consider \( \alpha \) values within the range 0.5-1.0 depending on site-specific soil data, spudcan/soil interface roughness, etc. An \( \alpha \) value less than 0.5 should be considered for situations such as a hard clay at the surface. In this case, the standard form of the yield surface should be considered. The standard form can also be used when it provides more favourable results.

**NOTE** << The last sentence was added because the present formulation can cut off useful volume because it passes through \( V/V_{Lo} = 0.5 \). The revised surface should depart from the standard surface at a tangent such that it always increases the volume. It is intended that, if derived, an improved formulation will be incorporated here, and into the general equation (Edwards has some suggestions). Otherwise, the last sentence will be updated to permit the tangent approach. >>

The expression above can be re-written to give the maximum spudcan moment as a function of the horizontal and vertical forces. For a vertical and horizontal force combination that lies inside the yield surface given above, the spudcan moment is limited to the maximum available moment capacity \( Q_M \):

\[
F_M = f_2Q_M \left[ 1 - \left( \frac{F_H}{f_1Q_H} \right)^2 \right]^{0.5}
\]

(A.9.3-25)

The equation above only applies when:

\[
0.0 < F_V < 0.5Q_V
\]

and
A.9.3.3.4 Extension of the yield surface for additional penetration in sand

On seabeds of silica sands, conical spudcans that are not fully seated can develop increased moment capacity due to further penetration and the resulting increase in contact area. The effect can be taken into account for spudcans with \( F/Q > 0.5 \). The moment capacity \( Q_{M_{ps}} \) associated with further penetration is estimated as the minimum of \( Q_{M_{ps}} \) and \( Q_{M_{pv}} \), calculated as follows (Svanø and Tjelta, Reference [A.9.3-49]):

\[
Q_{M_{ps}} = 0.075 B Q_{V} (B_{max}/B)^3
\]

(A.9.3-26)

\[
Q_{M_{pv}} = 0.15 B F_{V}
\]

(A.9.3-27)

in which \( B \) is the plan diameter of the effective contact area after preload, and \( B_{max} \) is the plan diameter of the maximum possible contact area i.e. when the spudcan is fully seated.

NOTE << There is a query for the Technical Panel as to whether limits should be given on the cone angle. >>

The combined capacity should be checked against the modified yield interaction surface:

\[
\frac{F_{H}}{Q_{H}} \times \left( \frac{F_{M}}{Q_{M}} \right)^{2} - 16 \left( \frac{F_{V}}{Q_{V}} \right)^{2} \left( 1 - \frac{F_{V}}{Q_{V}} \right)^{2} = 0
\]

(A.9.3-28)

Additional penetration of spudcan in sands can also 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, Reference [A.9.3-50]; Cassidy et al., Reference [A.9.3-51] and Bienen, et al., Reference [A.9.3-47].

A.9.3.3.5 Extension of the yield surface for additional penetration in clay

For additional penetration of spudcans in clay, References [A.9.3-34] and [A.9.3-37] provide work-hardening modifications to the yield surface equations. Updated stiffnesses and capacities are determined through plasticity principles.

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 rigid disk on an elastic half-space with modification factors to account for spudcan embedment. For the effects of leg embedment see A.9.3.4.5. 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:

Vertical stiffness \( K_1 = K_{d1} \frac{2GB}{(1-v)} \)

(A.9.3-29)

Horizontal stiffness \( K_2 = K_{d2} \frac{16GB(1-v)}{(7-8v)} \)

(A.9.3-30)

Rotational stiffness \( K_3 = K_{d3} \frac{GB^3}{3(1-v)} \) (from Winterkorn[A.9.3-52]) for relatively low levels of loading)
The selection of the shear modulus, \( G \), is discussed later in this Clause. An upper or lower bound value should be selected as appropriate for the analysis being undertaken. 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.

A.9.3.4.2 Stiffness modifications

A.9.3.4.2.1 Embedment

Table A.9.3.1 provides values for the 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, Bell\[^{A.9.3-53}\].

<table>
<thead>
<tr>
<th>2D/B</th>
<th>( K_{d1} )</th>
<th>( K_{d2} )</th>
<th>( K_{d3} )</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Open hole</td>
<td>With backfill</td>
<td>Open hole</td>
</tr>
<tr>
<td>0,0</td>
<td>1,00</td>
<td>1,00</td>
<td>1,00</td>
</tr>
<tr>
<td>0,5</td>
<td>1,15</td>
<td>1,21</td>
<td>1,33</td>
</tr>
<tr>
<td>1,0</td>
<td>1,28</td>
<td>1,41</td>
<td>1,44</td>
</tr>
<tr>
<td>2,0</td>
<td>1,42</td>
<td>1,70</td>
<td>1,51</td>
</tr>
<tr>
<td>4,0</td>
<td>1,59</td>
<td>2,00</td>
<td>1,61</td>
</tr>
</tbody>
</table>

A.9.3.4.2.2 Cyclic loading

According to Andersen\[^{A.9.3-54}\], for clays susceptible to cyclic degradation (i.e. with OCR \( \geq 4 \)) cyclic degradation reduces the horizontal bearing capacity by 30%, i.e. the horizontal bearing capacity calculated from static soil properties should be multiplied by a reduction factor \( = 0.7 \). The cyclic soil stiffnesses calculated from static soil stiffnesses can be multiplied by factors of 1,25 horizontal, 1,25 rotational and 3 to 8 vertical. The reference static soil stiffnesses are first loading, small strain values, not including unload/reload effects.

A.9.3.4.2.3 Linear vertical, linear horizontal and secant rotational stiffness

Except for simple dynamic analyses with linearized foundations, contained within A.10.4.4 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 of \( (F_V, F_H, F_M) \) 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 of \( (F_V, F_H, F_M) \) lies inside the yield surface, the initial estimate of linearized rotational stiffness should also be reduced by the following procedure using the factor, \( f_r \), which has an increasing effect as the yield surface is approached. The rotational stiffness reduction factor is given by the following expression, Templeton\[^{A.9.3-55}\]:

\[
f_r = \frac{1}{1 - n} \frac{n}{\ln[(1 - nr) / (1 - n)]}
\]

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 best available data (either empirical or analytical) applicable to the jack-up and location. Finite element analysis for Gulf of Mexico clay indicates 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 stiffness reduction expression takes a simpler form,

\[
f_r = n / \ln(1 - n)
\]

(A.9.3-32)

As \( n \) approaches 1,0 the stiffness reduction expression tends towards the form below, which gives the most conservative treatment of stiffness reduction.

\[
f_r = 1 - n.
\]

(A.9.3-33)

The variable, \( n \), in the stiffness reduction expression is the failure ratio defined by:

\[
r_f = \left\{ \frac{F_H}{Q_h} \right\}^2 + \left\{ \frac{F_M}{Q_M} \right\}^2 \leq 1,0
\]

(A.9.3-34)

where \( a \) is as defined in A.9.3.3.2 above.

NOTE \( n > 1,0 \) 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 \( F_V/Q_V < 0.5 \), the failure ratio can be expressed as:

\[
r_f = \left\{ \frac{F_H}{f_1 Q_h} \right\}^2 + \left\{ \frac{F_M}{f_2 Q_M} \right\}^2 \leq 1,0
\]

(A.9.3-35)

where \( f_1 \) and \( f_2 \) are as defined in A.9.3.3.3 above.

NOTE << Equation A.9.3-35 will need to be revised if an improved format is determined for Equation A.9.3-22. >>

A.9.3.4.2.4 Nonlinear vertical, horizontal and rotational stiffness

The full interaction surface model, which includes nonlinear vertical, horizontal and rotational stiffnesses, implicitly incorporates the necessary stiffness reduction as a consequence of work hardening plastic displacement and rotation. The stiffness reduction factor should not be applied.

A.9.3.4.2.5 Nonlinear continuum foundation model

A continuum foundation model which includes nonlinear soil behaviour (e.g., elastic-plastic work hardening) implicitly incorporates the necessary stiffness reduction. The stiffness reduction factor should not be applied.

A nonlinear continuum foundation model should not be used unless one of the simpler analysis methods above has been used to provide a benchmark for the results.

A.9.3.4.3 Selection of Shear Modulus, \( G \), in clay

The value of the initial, small-strain shear modulus for clay should be based on the value of the shear strength \( (s_u) \) measured at the depth \( z = D + 0.15B \) where \( B \) is the largest diameter of the spudcan in contact with the soil and \( D \) is the predicted depth below sea floor of the lowest point on the spudcan with diameter \( B \). Where the clay is significantly layered the average strength within the range \( z = D \) to \( z = D + 0.3B \) should be used. Except in areas with carbonate clays or clayey silts the shear modulus should be calculated as \( G^\text{[A.9.3-56].} \)
\[ G = \frac{s_u \times 600}{O^{0.25}} \quad \text{with } G < s_u I_{NC} \text{ and subject to the limitations given below. (A.9.3-36)} \]

where

\[ O = \text{the overconsolidation ratio}; \]
\[ I_{NC} = \text{rigidity index for normally consolidated clays} \]

For extreme loading situations, and in the absence of other data, \( I_{NC} \) should be conservatively limited to 400, Noble Denton\(^\text{[A.9.3-56]}\).

NOTE The recommendations of [A.9.3-56] are based on overconsolidated clays with plasticity indices of up to 60%. 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%.

\[ I_{NC} = 600 \] is supported by field data for jack-up response in the Gulf of Mexico, Templeton et al\(^\text{[A.9.3-38]}\).

It should be noted that \( I_{NC} \) is inversely proportional to the plasticity index (Andersen, Reference [A.9.3-54], Figure 10.2, reproduced below). For clays with plasticity indices in excess of 60%, and not covered by field data, the analyst should account for this inverse relationship when determining \( G \).

In some cases higher ratios of \( I_{NC} \) may be used. The data published by Andersen\(^\text{[A.9.3-54]}, \) Figure 10.2, would support use of values as high as 1000 or even 2500, particularly for plasticity indices less than 20%.

The recommendations given above 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 \( G \) values is degraded, either explicitly in the linearized foundation model using the stiffness reduction formulae given in A.9.3.4.2.1, or implicitly using nonlinear 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 upper-bound values of \( G \).
A.9.3.4.4 Selection of shear modulus, $G$, in sand

For sands, the initial small-strain shear modulus should be computed from:

$$
\frac{G}{\rho_u} = j \left( \frac{V_{swl}}{\rho_a} \right)^{0.5}
$$  \hspace{1cm} (A.9.3-37)

where

- $j = \text{dimensionless stiffness factor}$
- $\rho_u = \text{atmospheric pressure}$
- $D_R = \text{relative density (percent)}$
- $V_{swl} = \text{seabed vertical reaction under still water conditions.}$
The recommendations given above 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 $G$ values is degraded, either explicitly in the linearized foundation model using the stiffness reduction formulae given in A.9.3.4.2.1, or implicitly using nonlinear 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 upper-bound values of $G$.

### A.9.3.4.5 Selection of shear modulus in layered soils

Roesset, Reference [A.9.3-57], provides equations 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, Reference [A.9.3-58].

### 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[A.9.3-59]).

The lateral soil resistance can be modelled based on concepts proposed by Matlock[A.9.3-60] 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 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.

NOTE << The limitation “Any face of each leg in compressive contact” may be revised/clarified. >>

### A.9.3.5 Vertical-horizontal capacity envelopes

#### A.9.3.5.1 General ultimate vertical-horizontal bearing capacity envelope

The general gross ultimate vertical-horizontal bearing capacity envelope for jack-up spudcans is a two-dimensional slice of the full vertical-horizontal-moment envelope given in A.9.3.3.2. If the spudcan moment capacity is zero (i.e. $F_M = 0$), the ultimate vertical-horizontal bearing capacity envelope is:

$$\left(\frac{F_H}{Q_H}\right)^2 - 16(1 - a) \left[\frac{F_V}{Q_V}\right]^2 \left[1 - \frac{F_V}{Q_V}\right]^2 - 4a \left[\frac{F_V}{Q_V}\right] \left[1 - \frac{F_V}{Q_V}\right] = 0 \quad (A.9.3-38)$$

For small embedments (in the limit as $a \to 0$), this equation reduces to:

$$\left(\frac{F_H}{Q_H}\right)^2 - 4 \left[\frac{F_V}{Q_V}\right] \left[1 - \frac{F_V}{Q_V}\right] = 0 \quad (A.9.3-39)$$

where $Q_V$ is taken to be equal to the gross ultimate vertical bearing capacity of the soil beneath the spudcan (achieved during preloading), evaluated as described in A.9.3.2.2.5, and $Q_H$ as defined in A.9.3.3.2.

#### A.9.3.5.2 Ultimate vertical-horizontal bearing capacity envelopes for spudcans in sand

The yield surface used for checking the vertical-horizontal bearing capacity of spudcans in sand is presented in A.9.3.5.1. The yield surface used for checking the sliding capacity of a spudcan in sand is given by:
\[ Q_H = F_V \tan \delta + 0.5 \gamma' (k_p - k_a) (h_1 + h_2) A_s \]  

(A.9.3-40)

where

\[ F_V = V_{st} \quad \text{no backfill} \]

\[ = V_{st} + W_{BF,0} + W_{BF,A} - B_S \quad \text{with backfill} \]

\[ h_1 = \text{embedment depth to the uppermost part of the spudcan, (if not fully embedded = 0).} \]

\[ h_2 = \text{spudcan tip embedment depth} \]

\[ k_a = \text{active earth pressure coefficient (for } x_u = 0) = \tan^2(45 - \phi/2) \]

\[ k_p = \text{passive earth pressure coefficient} = 1/k_a \]

\[ \delta = \text{the steel/soil friction angle (degrees).} \]

\[ \delta = \phi - 5^\circ \quad \text{for a flat plate, and} \]

\[ \delta = \phi \quad \text{for a rough surfaced conically shaped spudcan.} \]

A.9.3.5.3 Ultimate vertical-horizontal bearing capacity envelopes for spudcans in clay

The yield surface used for checking the horizontal and vertical bearing capacity for spudcans in clay for \( F_V > 0.5 Q_V \) is presented in A.9.3.5.1.

The sliding capacity in clay when \( 0 \leq F_V \leq 0.5 Q_V \) can be assumed to be \( Q_H \) as defined in A.9.3.3.2.

A.9.3.5.4 Ultimate vertical-horizontal bearing capacity envelopes for spudcans on layered soils

The bearing 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 through 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.9.3-13 shows the overall approach to the foundation acceptance checks.
A.9.3.6.2 Level 1, Step 1a: Ultimate bearing capacity for vertical loading - Preload check

The preload check should only be applied when the leeward leg horizontal reaction, $F_H$, is small (see table A.9.3-5 below). In this case the minimum preload, $V_{Lo}$, is obtained from:

**NOTE** << The above Figure needs to be further updated so it is fully compatible with A.9.3-1 >>
\[ V_{st} + W_{BF,A} \leq V_{Lo} / \gamma_{R,PRE} \]  

(A.9.3-41)

**NOTE**  The equation above can also be written as: \( F_v - W_{BF,o} + B_S \leq V_{Lo} / \gamma_{R,PRE} \)

where

\[ \gamma_{R,PRE} = \text{the preload resistance factor}, \]
\[ = 1,1 \]
\[ W_{BF,A} = \text{the submerged weight of any backflow and infill that is predicted to occur after preloading.} \]
\[ F_v = \text{gross bearing reaction under storm conditions;} \]
\[ = V_{st} \text{ no backfill} \]
\[ = V_{st} + W_{BF,o} + W_{BF,A} - B_S \text{ with backfill} \]
\[ V_{st} = \text{the structural vertical reaction beneath the spudcan due to the factored actions as determined from the procedures given in Clause 10, including leg weight and water buoyancy but excluding the submerged weight of backfill \( (W_{BF,o} + W_{BF,A}) \) and spudcan soil buoyancy \( (B_S) \).} \]

**NOTE**  The soil buoyancy of the spudcan in the formulation above is a simplification based on the spudcan either being completely buried or not buried at all. A more precise formulation can be used to account for partial embedment.

<table>
<thead>
<tr>
<th>Soil Type</th>
<th>Embedment</th>
<th>Maximum available horizontal sliding reaction, ( Q_{Hs} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sand</td>
<td>Partial</td>
<td>([ 0,1 - 0,07(B/B_{max})^2 ] F_v )</td>
</tr>
<tr>
<td></td>
<td>Full</td>
<td>0,03F_v</td>
</tr>
<tr>
<td>Clay</td>
<td>Less than 1,0 spudcan diameter</td>
<td>(( 0,16 + 0,9 \ (D/B)(A_s/A) ) ) F_v</td>
</tr>
<tr>
<td></td>
<td>1,0 spudcan diameter or greater</td>
<td>1,3 ( F_v A_s / A )</td>
</tr>
</tbody>
</table>

**NOTE**  The technical work Templeton[^A.9.3-61] supports use of 1,3 \( F_v A_s / A \) at embedment from approximately 1 to 2 times the diameter for clay.

When the leeward leg horizontal reaction, \( F_H \) determined from the procedures given in Clause 10, is small, the ultimate vertical bearing capacity is assumed to be the same as the maximum reaction beneath the spudcan during preloading \( (V_{Lo}) \) and the available reaction is \( V_{Lo} / \gamma_{R,PRE} \).

In sand with a vertical reaction of 0,9\( V_{Lo} \), the available horizontal capacity is approximately 0,03\( V_{Lo} \). If the horizontal reaction exceeds 0,03\( V_{Lo} \) additional penetration can occur. 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.

Where the spudcan is partially embedded (i.e., with maximum bearing area 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 horizontal capacity to 0,1\( V_{Lo} \).
A.9.3.6.3 Level 1, Step 1b: Ultimate sliding capacity check of windward leg

The horizontal capacity of the foundations of the windward leg(s) should be checked for the horizontal leg reaction \( F_H \) in association with the vertical leg reaction \( F_V \) determined from the procedures given in Clause 10. 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, however it is good practice to assess the horizontal capacity for all legs and load cases.

The foundation should satisfy the following capacity check:

\[
F_H \leq Q_H / \gamma_{R,Hfc}. \tag{A.9.3-42}
\]

where

\[
Q_H = \text{foundation capacity to withstand horizontal forces (A.9.3.5.2 to A.9.3.5.4)}
\]

\[
\gamma_{R,Hfc} = \text{partial resistance factor for horizontal foundation capacity.}
\]

- 1.25 (effective stress - sand/drained).
- 1.56 (total stress - clay/undrained).

A.9.3.6.4 Level 2, Step 2a: Bearing capacity/sliding check - Pinned spudcan

A reduction in vertical bearing capacity, \( Q_V \), of a spudcan foundation occurs when it is simultaneously subjected to horizontal force, \( F_H \), and moment, \( F_M \). The latter is ignored in Step 2a analyses as the spudcans are considered to be pinned.

The vertical/horizontal yield surface for sands and clays can be generated according to A.9.3.5 and the spudcan reactions should be evaluated for each spudcan. If the environmental response spudcan reaction lies within the factored yield surface the foundation is satisfactory. To obtain the factored yield surface, the capacities are divided by the resistance factor \( \gamma_{R,VH} \). A measure of the utilization can be obtained by assessing the proximity of the reaction points to the factored yield surface. When making the check, the magnitude of the vector to the factored yield surface should be compared against the magnitude of the vector to the environmental response point, \( (F_V,F_H)_{ORG} \). To ensure that the magnitude of the resulting utilization is meaningful, the origin of the vectors \( (F_V,F_H)_{ORG} \) should be taken on the vertical capacity axis (at zero shear) at \( 0.5V_{Lo} / \gamma_{R,VH} \) (see Figure A.9.3-14) and each spudcan foundation should satisfy the following capacity check:

\[
| (F_V,F_H) - (F_V,F_H)_{ORG} | \leq | (Q_{VH}) - (F_V,F_H)_{ORG} | / \gamma_{R,VH} \tag{A.9.3-43}
\]

where

\[
Q_{VH} = \text{point where the vector originating from the still water and passing through } (F_V,F_H) \text{ intersects the applicable vertical and horizontal capacity surface from A.9.3.5.}
\]

\[
\gamma_{R,VH} = \text{partial resistance factor for foundation capacity.}
\]

- 1.1 - Maximum bearing area not mobilized.
- 1.15 - Penetration sufficient to mobilize maximum bearing area.

\[|...| = \text{vector magnitude}\]

The spudcan foundations should also be assessed using the sliding check given in 9.3.6.3 since the factored sliding failure surface can lie within the factored vertical/horizontal capacity envelope.
NOTE 1 The selection of the origin at \(0.5 V_{Lo} / \gamma_{R,VH}\) with zero shear is arbitrary. This origin establishes a mechanism for developing the utilization.

NOTE 2 Where the full bearing area is mobilized, the resistance factor is larger because the rate of gain in capacity is attributable only to the soil, whereas with partial mobilization the area also increases.

\[
U = \left( F_V, F_H \right) / \left( Q_{VH} / \gamma_{R,VH} \right)
\]

\[
0.5 V_{Lo} / \gamma_{R,VH}
\]

**Figure A.9.3-14** Vertical-horizontal bearing capacity envelope

**A.9.3.6.5 Level 2, Steps 2b and 2c: Spudcan with moment fixity and vertical and horizontal stiffness**

Step 2b and 2c response analyses inherently include a check on compliance with the unfactored foundation yield surface except that in a Step 2b analysis compliance is no longer assured when the moment fixity has reduced to zero i.e. the spudcan has become pinned.

The capacity checks to be undertaken in a Step 2b assessment are identical to those undertaken for Step 2a using the vertical and horizontal reactions from the response analysis which has accounted for spudcan moment fixity with stiffness reduction.

A Step 2c analysis implicitly includes a check on compliance with the unfactored foundation yield surface. When the frictional sliding surface intersects the yield surface, sliding can occur before the response reaches
the yield surface. When this sliding effect is included in the response analysis, no Level 2 checks are required. When this sliding effect is not included, a sliding check should be undertaken in accordance with A.9.3.6.3. In all cases, the Level 3 Step 3a displacement check should be performed.

A.9.3.6.6 Level 3, Steps 3a and 3b: Settlements resulting from exceedence of the foundation capacity

Vertical settlement and/or sliding of a spudcan can occur if the reaction due to storm loading is 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.9.3.2.6 and A.9.3.2.7.

A Step 3a check can be accomplished by identifying the "virtual" preload level that would be required to expand the $V-H$ yield surface used in Step 2 such that the factored capacity exceeds the spudcan foundation reactions. The displacement associated with this "virtual" preload is then obtained from the load-penetration curve. If the displacement is significant, the effects on the spudcan foundation reactions 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 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 reactions 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 location. The limits are dependent upon the jack-up and the configuration at the location.

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 due to bearing capacity failure during to preloading are discussed in A.9.3.2. 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. The elastic settlements occur concurrently with applied actions and can be calculated as function of the basic elastic soil properties ($\nu$ 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.9.4.6).
A.9.3.7 Skirted spudcans

No guidance offered.

A.9.4 Other considerations

A.9.4.1 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, for example:

— This 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 & guides can cause overstress.

Anticipated installation-induced stresses (e.g. due to a sloping seabed) should be included in the overall site-specific assessment. If the installation assumptions are less conservative than the as-installed condition (e.g. due to differential jacking on each chord of the leg), the assessment should be updated accordingly.

Consideration can be given to the potential benefit of seabed preparation prior to emplacement of the jack-up.

A.9.4.2 Footprint considerations

Surface or buried footprints from prior jack-up operations at the proposed location 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 References [A.9.4-1] to [A.9.4-8].

A.9.4.3 Leaning instability

Leaning instability of jack-ups can occur during preloading operations in soft clays where the rate of increase in bearing capacity with depth is small. A lower bound estimate of the leaning stability can be obtained using the theory of Hambly, Reference [A.9.4-9]. 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.

In deep water 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.9.4.4 Leg extraction difficulties

Leg extraction difficulties can be caused by conditions including:

— 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 could affect the future emplacement of jack-ups at the same location.

Further details can be found in References [A.9.4-10] to A.9.4-15

A.9.4.5 Cyclic mobility

General guidance on the assessment of the potential for liquefaction and/or cyclic mobility is given by Kramer[A.9.4-16] and Idriss and Boulanger[A.9.4-17]. Dean[A.9.4-18] presents approximate methods for estimating settlements of submerged foundations subjected to time dependent loading.

References:

A.9.4.6 Scour

The key conditions for scour are:

— hydrodynamic conditions

— flow disturbance due to presence of an obstruction

— the 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, either the spudcan itself or the spudcan in combination with the leg structure.

Particle size has a strong influence on the erodibility, see Figure A.9.4-1. 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:

— 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), Reference [A.9.4-19]; Whitehouse (1998), Reference [A.9.4-20] and Rudolph
et al., Reference [A.9.3-21]. 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:

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 can be evaluated from scour surveillance monitoring.

c) Monitoring and adjusting for reduction in hull elevation.

![Image of soil particle size and seabed mobility](Figure A.9.4-1 Soil particle size and seabed mobility)

**Figure A.9.4-1 Soil particle size and seabed mobility**

### A.9.4.7 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. 1991[A.9.4-22], Stewart (Perth, 2005) [A.9.4-23], NUS (Perth, 2005) [A.9.4-24], and GEO/DGI[A.9.4-25].

### A.9.4.8 Geohazards

Certain areas of the world, including the US, require shallow geohazard surveys and publish documents that can give some useful guidance e.g. Reference US MMS[A.9.4-26] and OGP[A.9.4-27]. 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.
A.9.4.9 Carbonate material

NOTE << Text and bibliographic references are to be added >>

A.10 Structural response

A.10.1 Applicability

No guidance offered.

A.10.2 General considerations

The ULS responses typically include the internal forces in the leg members, overturning moments of the jack-up, horizontal deflections of the hull, reactions and displacements at the spudcans 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 required by the acceptance criteria in Annex B. The application of actions is described in A.8.8. Clause 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 loading(s) for each item in the list above is (are) determined.

When determining the FLS response, the cumulative number of stress cycles is 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 deterministic quasi-static analysis procedure including an inertial loadset (see A.10.5.2) or by a more detailed fully integrated (random) dynamic analysis procedure which uses the stochastic method (see A.10.5.3). Figure A.10.3-1 shows the procedures for using these two approaches to determine the ULS response.
Figure A.10.3-1 — Recommended approach to determine extreme responses
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.

A.8 identifies the factors that affect the structural stiffness of the jack-up and discusses the structural stiffness modelling at various levels of complexity. The hydrodynamic and wind actions are discussed in Clause 7.

The magnitude of the dynamic response is affected by:

a) The dynamic characteristics (natural periods) of the structural system formed by the jack-up on its foundation, and

b) The characteristics of the wave/current excitation. For 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 following sections discuss the factors that affect these two characteristics.

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 the following equation:

$$ T_n = 2\pi \sqrt{\frac{M}{K}} $$

where

- $T_n$ = highest (or first mode) natural period
- $M$ = effective system mass
- $K$ = effective system stiffness.

C.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:

1) bending deformation of the legs;
2) shear deformation of the legs;
3) axial deformation of the legs;
4) hull bending deformation;
5) horizontal vertical and rotational leg-hull connection stiffness;
6) horizontal, vertical and rotational foundation stiffness;
7) second order $P-\Delta$ due to lateral displacement of the hull;
8) Euler amplification of local member forces.

The model can contain a number of nonlinear elements, notably the leg to hull connections and the spudcan-foundation interfaces.

If desired the system stiffness for the fundamental modes may be determined from an idealized single degree-of-freedom system as described in Annex C.

A.10.4.2.3 Mass

See 8.7 for guidance.

A.10.4.2.4 Variability in natural period

Due to uncertainty in parameters affecting the natural period, the calculated natural period(s) can also be uncertain. 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 combined response is maximised not just the dynamic amplification since it is possible to have a large DAF combined with a low metocean loading, due to cancellation, leading to low combined response. 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 30m. The potential for increased response due to shortcrested waves should be considered (see A.7.3.3.3.3). For further details refer to C.3.
Figure A.10.4-1 — Periods for wave force cancellation and reinforcement as a function of leg spacing
A.10.4.3 Damping

A.10.4.3.1 General

The main components of system damping are foundation, hydrodynamic and structural. Each of these can be modelled either linearly or nonlinearly 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 nonlinear 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. The small strain soil material damping is typically small. At larger strains, amplitude-dependent hysteretic damping can also occur. Where a nonlinear 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.

NOTE << There has been a comment from Jack Templeton that hysteretic damping should be allowed with compensating reduction in linearised stiffness. He suggests replacing last sentence above with: "Where linearized soil stiffness is used in a time domain analysis, and rotational stiffness is limited, per A.10.4.4.1, to 80-100% of the value determined from the formulation in A.9.3.4.1, 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, then the total linear system damping should not exceed 7% without credible, applicable justification. Lower values can be appropriate for fatigue analyses and smaller 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

<table>
<thead>
<tr>
<th>Damping source</th>
<th>Global linear damping not to exceed (% of critical damping)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Structure, holding system, etc.</td>
<td>2%</td>
</tr>
<tr>
<td>Small strain foundation</td>
<td>2% [*]</td>
</tr>
<tr>
<td>Hydrodynamic</td>
<td>3% or 0% [*]</td>
</tr>
</tbody>
</table>

1. The small-strain soil material damping is typically small; in the absence of specific data, 2% is considered to be a reasonable estimate.

2. In cases where the relative velocity formulation is used (α = 1 in Equation A.7.3.3.3) 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

When a reduction factor is being applied to reduce the spudcan rotational stiffness used for determining the DAF (see A.10.5.2.2), an additional damping allowance may be made to account for the effects of foundation hysteretic damping. See C.4.

A.10.4.3.4 Vertical radiation damping in earthquake analysis

NOTE << Some updates to the following text are expected to be offered by V. Karthigeyan & Jack Templeton. >>
In earthquake analyses the foundation (wave propagation) radiation damping can be included for vertical motion of the spudcan in addition to other foundation damping. The 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. In simpler analyses, the vertical foundation radiation damping can be estimated from the work of Lysmer and Richart, Reference [A.10.4-1], from the following equation:

\[ C_{rd} = [0.85 \frac{B^2}{(1 - \nu)}] \sqrt{(G \rho)} \tag{A.10.4-1} \]

where

- \( C_{rd} \) is the radiation damping dashpot (force per unit velocity)
- \( B \) is the equivalent diameter of the spudcan
- \( \nu \) is Poisson's ratio of the foundation soil
- \( G \) is the shear modulus of the foundation soil
- \( \rho \) is the total, saturated, (mass) density of the foundation soil.

In nonlinear dynamic analyses, or in linear time domain dynamic analyses using direct integration, this equation can be used directly to establish the value for the foundation dashpots.

In linear modal dynamic analyses, the additional contribution of vertical radiation damping to the linear damping ratio for the primary surge, sway and vertical modes only can be calculated from the following equation:

\[ \zeta_{rd} = 0.232 \ast B \ast \sqrt{(\rho/G)} \ast \omega_n \tag{A.10.4-2} \]

where

- \( \zeta_{rd} \) is the radiation modal damping ratio to account for spudcan vertical motion
- \( \omega_n \) is the natural frequency (radians).

NOTE 1 Equation A.10.4-1 was obtained by combining the definition of the damping ratio with equation A.10.4-2 for damping coefficient, \( C' \), and the corresponding equation for stiffness given by Lysmer and Richart (1966), Reference [A.10.4-1].

NOTE 2 Within the ranges of typical spudcan sizes and soil conditions, expected values of the linear damping ratios for radiation damping are typically less than 1% in the case of natural periods likely to be excited significantly by wave action. Such values can be ignored in comparison to the damping being accounted from other sources. However, in the case of earthquake actions, the expected values of radiation damping ratios can be large (>10%). Radiation damping values this large would 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

The analysis of the structure and the foundation evaluation can be performed in two different ways:

Option 1: Deterministic two-stage approach. The first stage, is to calculate the dynamic amplification and inertial loadset, often using linearised 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 nonlinear time-domain stochastic analysis is performed taking into account the elasto-plastic behaviour of the spudcan stiffness.

A.10.4.4.1.2 Option 1 - deterministic two-stage

In this approach the dynamic excitation of the structure is evaluated based on either a simple linear analysis or a more complex elasto-plastic analysis. The foundation and structural assessment is then performed using a quasi-static iterative analysis technique, for which the dynamic actions have already been determined. The dynamic analysis can include linearized foundation fixity. Typically the initial linearized rotational stiffness for the dynamic analysis can be taken as 80-100% of the value determined from the formulation in A.9.3.4.1. This simplified approach does not capture the temporary reductions in stiffness which occur during plasticity events (generally with detrimental effects), but also does not capture the increased damping associated with these events (with beneficial effects).

This quasi-static analysis can be accomplished by means of either an elasto-plastic foundation model or a simplified application of the full plasticity analysis as described below. This simple approach can be used to create moments on the spudcan by inclusion of a simple linear rotational spring. The moments thus induced on the spudcan are limited to a capacity based on the yield interaction relationship between vertical force \( F_V \), horizontal force \( F_H \) and moment \( F_M \) acting at the spudcan.

This simple procedure is described in the following steps:

1) Include vertical, horizontal and (initial) rotational stiffnesses (linear springs) (see A.9.3.4.1) to the analytical model and apply the factored functional and factored metocean actions together with the associated and separately calculated inertial actions from a linearized dynamic analysis.

2) Calculate the yield interaction function value (see A.9.3.3) using the resulting forces at each spudcan. If the results indicate that the force combination falls outside the yield surface, reduce the rotational stiffness (arbitrarily) and repeat the analysis.

3) Repeat step 2 until the force combination at each spudcan lies essentially on the yield surface. If 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.

4) 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.

A.10.4.4.1.3 Option 2 - stochastic one-stage

In this approach the dynamic structural analysis and assessment is performed using one model. A fully detailed nonlinear time-domain analysis is performed taking into account the elasto-plastic behaviour of the foundation.

The effects of the foundation fixity are simultaneously considered on both the dynamic response and the seabed reactions. 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 dynamic analysis to determine structural response and foundation forces at each time step.

b) Compute the foundation behaviour using a nonlinear 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 analyses should be performed for differing wave histories, and the MPME's 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 (see A.10.4.2.1).

A.10.4.4.2 Foundations for earthquake assessment

For the simple screening assessment, the foundation should be modelled with the maximum interpreted shear modulus from clause 9, without degradation and with appropriate rate adjustments.

For more detailed assessments a fully nonlinear coupled yield interaction model or a continuum model should be used with degradation effects.
Figure A.10.4-2 — Analysis procedure for two-stage assessment with foundation fixity (Option 1)
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 may generally be represented as a steady flow of air. The periods of waves typically lie between 3 sec and 20 sec. Since typical jack-up natural periods fall within this range, the primary source of dynamic excitation is from waves.

Sea waves are generally not regular but random in nature unless swell is predominant. 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 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 analysis ($K_{\text{DAF,SDOF}}$) or random wave time-domain (stochastic) analyses ($K_{\text{DAF,RANDOM}}$). In the second stage, the maximum quasi-static wave action is determined by ‘stepping’ the maximum wave through the structure. The maximum wave 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 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 (DAF) and inertial loadsets

A.10.5.2.2.1 General
When using a deterministic analysis for calculating the response, the dynamic response is represented by equivalent inertial actions as described in A.8.8.5. The inertial loadset can be derived from the simple SDOF approach 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. It should be noted that dynamic amplification is the result of the inertial response which is dominated by the hull mass. Therefore amplifying the hydrodynamic actions is not a correct representation.

$$R_{\text{DYNAMIC}} = R_{\text{MEAN}} + K_{\text{DAF,SDOF}}( R(t)_{\text{STATIC}} - R_{\text{MEAN}} ) \quad \text{or} \quad R_{\text{INERTIAL}} = ( K_{\text{DAF,SDOF}} - 1)( \text{Amplitude of } R(t)_{\text{STATIC}} )$$

(a) SDOF

$$K_{\text{DAF,RANDOM}} = \frac{MPM_{\text{DYNAMIC}}}{MPM_{\text{STATIC}}}$$

(b) Stochastic/Random

Figure A.10.5-1 — Dynamic amplification factors
NOTE This difference between the height of the 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 response.

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 mass-spring-damper mechanism. The (highest) natural period of the 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 single degree-of-freedom (SDOF) method is fundamentally empirical because (1) the wave/current action does not occur at the hull and (2) the loading is non-periodic (random) and nonlinear.

The method described below generally leads to an approximation of the jack-up’s real behaviour and has been calibrated against more rigorous methods. The following cautions are noted when using the SDOF method:

1) If the ratio of the jack-up natural period to the wave excitation period, $\Omega$, is less than 0.5 and the current velocity is small relative to the wave particle velocities, the SDOF method should give reasonably accurate results when compared to a more rigorous analysis.

2) If $\Omega$ is greater than 0.5, the relative position of the jack-up natural period within the base shear transfer function should be checked. As discussed in A.10.4.2, if the natural period falls near a wave force peak, then the SDOF method can be unconservative because it ignores forcing at other than the full wave excitation period. Note that a range of natural periods should be considered to account for a reasonable range of foundation fixity.

3) 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.

The ratio of (the amplitudes of the) dynamic to the quasi-static response as a function of frequency ($\omega$) or period ($T$) of steady state, periodic and sinusoidal excitation is calculated as the classical dynamic amplification factor ($K_{DAF,SDOF}$):

$$K_{DAF,SDOF} = \frac{1}{\sqrt{(1 - \Omega^2)^2 + (2\zeta\Omega)^2}}$$  \hspace{1cm} (A.10.5-1)

where

- $\Omega = \frac{\text{Jack - up natural period}}{\text{Wave excitation period}} = \frac{T_n}{T_w}$
- $\zeta = \text{damping ratio or fraction of critical damping} = (\% \text{ Critical Damping})/100 \leq 0.07$ (see A.10.4.3).
- $T_w = 0.9T_p$.
- $T_p = \text{most probable peak wave period}$.
- $T_n = \text{the jack-up natural period as derived in A.10.4.2.1}$.

The damping parameter $\zeta$ in this model represents the total of all damping contributions (structural, hydrodynamic and soil damping). For the evaluation of extreme response using the SDOF method a value not exceeding 0.07 is recommended.
The calculated $K_{DAF,SDOF}$ from the SDOF method is used to estimate an inertial loadset which represents the contribution of dynamics over and above the quasi-static response in accordance with Figure A.10.3-1. This inertial loadset should be determined as follows and applied at the hull centre of gravity in the direction of wave propagation:

$$F_{in} = (K_{DAF,SDOF} - 1) F_{BS,Amplitude} \tag{A.10.5-2}$$

where

- $F_{in}$ = magnitude of the inertial loadset.
- $F_{BS,Amplitude}$ = single Amplitude of quasi-static Base Shear over one wave cycle.
  $$= \frac{F_{BS,(QS)Max} - F_{BS,(QS)Min}}{2}$$
- $F_{BS,(QS)Max}$ = maximum quasi-static wave/current Base Shear.
- $F_{BS,(QS)Min}$ = minimum quasi-static wave/current Base Shear.

The above equation is part of a calibrated procedure and should not be altered. A more general inertial loadset procedure, using the results from random 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 random time domain dynamic analysis procedure, two DAF’s are calculated, one for the base shear (BS) and one for the overturning moment (OTM). The inertial loadset $F_{in}$ is calculated from these DAF’s. The BS and OTM DAF’s are the ratios of the MPME of the dynamic responses to the MPME of the static responses ($MPME_{dyn}/MPME_{static}$) determined from 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.10.5-2.

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 using negative springs to represent P-Δ, 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 normally 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, $F_{in}$, should be such that it increases both the BS and OTM responses of 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 does not need to 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.

### 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). The MPMEs of the base shear and overturning moment are used to develop DAF’s for use in the two-stage deterministic analysis. For a one-stage analysis the intent is to determine the MPME of the utilizations directly.

In all stochastic analyses all action factors are set to 1.0 (see 8.8.1.3). When the stochastic storm analysis is being 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 stochastic dynamic 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 that is fully described in A.7.3.3.3.3 which
identifies important constraints associated with this method of random wave dynamic analysis.

A.10.5.3.2 Application of partial factors to metocean parameters

When undertaking a one-stage fully integrated stochastic dynamic analyses, partial factors are applied to the
metocean parameters. To ensure consistency between the one-stage stochastic and two-stage deterministic
approaches, the partial factors should produce metocean load levels comparable to the factored quasi-static
metocean actions used in the deterministic method. When using stochastic dynamic analyses to determine a
DAF for application in a two-stage deterministic analysis, the partial factors should be set to unity.

The metocean partial factors for fully integrated one-stage stochastic dynamic analyses can be determined as
follows:

**Wind velocity partial factor:** 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 extremes are being used, or
- $\sqrt{1.25}$ if 100-year return period joint probability metocean data are being used.

**Wave height and current velocity partial factors:** The partial factors for wave height and current velocity to
be used in the stochastic analysis are determined through an iterative process. The process involves
factoring the wave height and current velocity until the partial-factored quasi-static stochastic wave/current
force matches the action-factored quasi-static deterministic wave/current force computed using higher-order
wave theory (see note below). The effects of wave spreading (see A.6.4.2.4) should be consistently included
or consistently excluded in the stochastic and deterministic loads used in the calibration. As a first
approximation, the same partial factors can be used as given above for wind. 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 and
- the stochastic mean and deterministic mean, the latter determined from integration over a full wave
cycle (i.e. not 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 seastate.

**NOTE** The reference level for the wave and current forces in the two stage approach 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 absolutely founded on the quasi static wave/current deterministic
action. Conversely, there is no simple equivalent in a fully integrated single stage analysis. It is therefore necessary to
determine a stochastic equivalent to the factored deterministic quasi static wave-current force. This is achieved by
calculating the stochastic loads over three hours until partial metocean factors are found which 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 equations 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 Table A.7.3-4.

Wave spreading may be taken into account either by using a 3-D analysis method or by using the kinematics reduction factor in a 2-D analysis (see A.6.4.2.4). Accounting for wave spreading generally results in a smaller DAF.

A.10.5.3.4 Methods for Determining the MPME

The extreme response used for the assessment is the Most Probable Maximum Extreme (MPME) response which has a 63% chance of exceedence in a 3-hour storm. This MPME response is defined in Table A.10.5-2 as the mode value or highest point on the Probability Density Function (PDF). The stochastic waves modelled using a random superposition model result in non-Gaussian responses. Three methods for obtaining the MPME of the response are included in Table A.10.5-2. It should be noted that the simpler modelling approaches do not lead directly to the MPME of all quantities of interest. For example, simpler multi-degree-of-freedom models provide the MPME of total leg forces, but do not lead directly to forces in individual members of a truss-leg. Fully integrated one-stage stochastic models can be used to determine the MPME for all assessment checks.

For two-stage random wave dynamic analysis the ratio of MPMEs of the dynamic to the quasi-static base reactions are used to determine the DAF that is used to calculate the inertial loadset (see A.10.5.2.2.3).

<table>
<thead>
<tr>
<th>Method</th>
<th>Recommendations</th>
</tr>
</thead>
<tbody>
<tr>
<td>General</td>
<td>Define the MPME as the extreme with a 63% chance of exceedence (typically this is the mode or highest point on the probability density function (PDF)). This is approximately equivalent to the 1/1000 highest peak level in a 3-hour storm and the extreme with approximately a 63% chance of exceedence.</td>
</tr>
<tr>
<td>Time Domain</td>
<td>Fit Weibull distribution to distribution, for 3-hour probability level. Take results as average of MPME's from at least 5 simulations. Each input wave simulation to be of sufficient length for recommendations of Table A.10.5-1 to be met (usually at least 60 minutes).  See C.5.1 or Use multiple 3-hour simulations and use Gumbel distribution on the extreme from each simulation. Sufficient simulations (usually at least 10) should be used to obtain stable MPME of responses.  See C.5.2. or Use Winterstein's Hermite polynomial model, with improvements by Jensen if the kurtosis is &gt; 5. Simulation of sufficient duration to provide stable skewness and kurtosis of responses (normally in excess of several hours). See C.5.3.</td>
</tr>
</tbody>
</table>

A.10.5.4 Initial leg Inclination

The effects of initial leg inclination should be considered. Leg inclination can occur due to leg-hull clearances and hull inclination. Generally hull inclination limits are set in the operations manual. The total horizontal offset due to leg inclination, \( \Omega_T \), can be estimated as:

\[
\Omega_T = \Omega_1 + \Omega_2 \quad \text{(A.10.5-3)}
\]

where

\[
\Omega_T = \text{Total horizontal offset of leg base with respect to hull.}
\]
If detailed information is not available, $O_T$ should be taken as 0.5% of the leg length below the lower guide.

The effects of leg inclination need be accounted for 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-∆ actions.

### A.10.5.5 Limit state checks

The ULS responses for assessment should be determined using appropriate combinations of actions due to fixed load 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 Clause 8.8. Clause 5.4.3 requires that the analysis is 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.10.5.3 is(are) determined. The checks cover:

#### Table A.10.5-3 — Action effects for limit state checks

<table>
<thead>
<tr>
<th>Limit State Check</th>
<th>Clause</th>
<th>Response Parameters(s)</th>
<th>Action Effect</th>
<th>$G_{V_{note 2}}$</th>
<th>$G$</th>
<th>$min$</th>
<th>$max$</th>
<th>$E_e$</th>
<th>$D_o$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Strength of members</td>
<td>12</td>
<td>Member force vectors$^3$</td>
<td>$G$, $V_{min, max}$</td>
<td>$E_e$</td>
<td>$D_e$</td>
<td>$Y^4$</td>
<td>$Y$</td>
<td>$Y$</td>
<td>$Y$</td>
</tr>
<tr>
<td>Overturning Stability</td>
<td>13.8</td>
<td>Overturning moment</td>
<td>$Y^5$</td>
<td>$Y^6$</td>
<td>$Y$</td>
<td>$Y$</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Foundation capacity:</td>
<td>13.9</td>
<td>Vertical leg reaction</td>
<td>$Y$</td>
<td>$Y$</td>
<td>$Y$</td>
<td>$Y$</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>- preload</td>
<td>A.9.3.6.2</td>
<td>Vertical &amp; Horizontal leg reactions</td>
<td>$Y$</td>
<td>$Y$</td>
<td>$Y$</td>
<td>$Y$</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>- sliding</td>
<td>A.9.3.6.3</td>
<td>Vertical, Horizontal ( &amp; moment) leg reactions</td>
<td>$Y$</td>
<td>$Y$</td>
<td>$Y$</td>
<td>$Y$</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>- bearing</td>
<td>A.9.3.6.4</td>
<td>Spudcan displacements and reactions</td>
<td>$Y$</td>
<td>$Y$</td>
<td>$Y$</td>
<td>$Y$</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>- displacement</td>
<td>A.9.3.6.6</td>
<td>Holding system force vectors</td>
<td>$Y$</td>
<td>$Y^4$</td>
<td>$Y$</td>
<td>$Y$</td>
<td>$Y$</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

$G$ = actions due to the fixed load

$G_V$ = actions due to maximum or minimum variable load

$E_e$ = metocean action due to the extreme storm event

$D_e$ = actions representing dynamic extreme storm effects

#### NOTES

1. In all instances the responses are evaluated including the effects of deformation under functional actions (hull sag) and large displacement (P-∆) effects.

2. Placed at most onerous centre of gravity position.

3. The effects of leg inclination to be added after global response analysis (see A.10.5.4).

4. Consider minimum variable load if this is more onerous.

5. Included in response calculation so P-∆ effects are captured.
### A.10.6 Fatigue

For long-term applications, refer to Clause 11.

### A.10.7 Earthquake

#### A.10.7.1 General

This clause complements ISO 19901-2 by presenting:

a) the assessment actions, combinations of actions and action effects resulting from ground motions, and

b) the approach to assessing a jack-up subjected to earthquake actions.

The greatest structural threat to a jack-up subject to an earthquake is likely to be from vertical excitations that result in uneven settlement of the spudcans which causes lateral instability of the jack-up. Table A.10.7-1 identifies the clauses within the normative and informative that provide guidance on earthquake modelling and analyses. A few general references to earthquake appear in other clauses.

**NOTE** In earthquake environments, operational issues (e.g. setback, rig clamping, drilling equipment) need special consideration to ensure that major hazards to personnel are mitigated.

<table>
<thead>
<tr>
<th>Normative Clause heading</th>
<th>Informative Clause heading</th>
</tr>
</thead>
<tbody>
<tr>
<td>6.6 Earthquake data</td>
<td>A.6.6 No guidance offered</td>
</tr>
<tr>
<td>7.7 Earthquakes</td>
<td>A.7.7 No guidance offered</td>
</tr>
<tr>
<td>8.6.3 Foundation modelling</td>
<td>A.8.6.3 Foundation modelling</td>
</tr>
<tr>
<td>8.7 Mass modelling</td>
<td>A.8.7 Mass modelling</td>
</tr>
<tr>
<td>8.8.1.1 Assessment actions / General</td>
<td>A8.8.8 No guidance offered</td>
</tr>
<tr>
<td>8.8.4 Earthquake analysis</td>
<td></td>
</tr>
<tr>
<td>9.4.6 Cyclic mobility</td>
<td></td>
</tr>
<tr>
<td>9.4.9 Geohazards</td>
<td>A.10.2 Structural response / Applicability / General considerations</td>
</tr>
<tr>
<td>10.3 Types of analyses and associated methods</td>
<td>A.10.4.3.4 Vertical radiation damping in earthquake analysis</td>
</tr>
<tr>
<td>10.7 Earthquake</td>
<td>A.10.7.1 General</td>
</tr>
<tr>
<td></td>
<td>A.10.7.2 Earthquake assessment procedure</td>
</tr>
<tr>
<td></td>
<td>A.10.7.3 ELE assessment</td>
</tr>
<tr>
<td></td>
<td>A.10.7.3.1 Partial action factors</td>
</tr>
<tr>
<td></td>
<td>A.10.7.3.2 Structural and foundation modelling</td>
</tr>
<tr>
<td></td>
<td>A.10.7.4 ALE assessment</td>
</tr>
<tr>
<td></td>
<td>A.10.7.5 Near source excitation</td>
</tr>
<tr>
<td>Annex B</td>
<td></td>
</tr>
</tbody>
</table>

#### Table A.10.7-1 – References to earthquake analysis
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 screening methodology is:

a) Determine earthquake actions using either the simplified earthquake action procedure or the detailed earthquake action procedure, as specified in ISO 19901-2 to develop the rock response spectra. Use of the simplified procedure (maps) for the initial screening of rigs is encouraged.

b) Evaluate earthquake activity and the associated response 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 1000-year return period on rock, adjust the spectral shape for the 1000 year event as described in 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) Demonstrate the performance of the jack-up using the ULS assessment procedures provided in Clause 13.

e) If the rig does not pass the simplified check, proceed to a more detailed assessment in accordance with A.10.7.4 using alternative analysis methods (10.9) and the 19901-2 ALE procedures.

Table A.10.7-1 – 19901-2 references to earthquake analysis

NOTE << Table from P5 to be added >>

A.10.7.3 ELE assessment

A.10.7.3.1 Partial action factors

The foundation, leg members and leg-hull connection should be assessed for the partial factored assessment actions defined in 8.8.1 for earthquake situations. The inertial action induced by the ELE ground-motion should be determined using dynamic analysis procedures such as response spectrum analysis or time history analysis.

NOTE Reference can be made to the Table in Annex B which contains all of the applicable factors to be used in 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 permanent actions \( G_r \),

- 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, 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 of appurtenances (A.7.3.2). For motions along the longitudinal axis of the structural members and appurtenances, the added mass may be neglected (except for spudcans).

The structural model should include the three-dimensional distribution of the stiffness and mass of the structure.

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.

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. Additional damping, including hydrodynamic or soil induced damping (hysteretic and radiation), should be substantiated by special studies. Small strain initial stiffness soil springs should be used to determine the natural periods.

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. For cohesive soils this information should be supplemented with the remoulded shear strength data. Depth to bedrock or a competent soil layer is required, and can be estimated from regional considerations.

Foundation performance should be determined on the basis of studies that consider the assessment actions. Except for the simplified screening analysis, the nonlinear 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, nonlinear dynamic time history or pushover analyses can be used to evaluate cumulative displacements and the resulting structural condition.

Vertical foundation loads should not normally exceed the preload. If the vertical foundation loads exceed the preload and the ULS Step 3 displacement check reveals the potential for excessive additional penetration, nonlinear 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 ELE/ULS screening assessment a site-specific nonlinear ALE can be used to demonstrate acceptability. This may be satisfied by a pushover analysis or time history analyses using ALE excitation.

Where substantial spudcan settlement or liquefaction is a possibility, a fully nonlinear 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., $T \geq 1$ sec.), and vertical motions equal to or greater than horizontal motions at shorter periods (e.g., $T \leq 0.3$ sec.).

A.10.8 Accidental situations

No guidance is offered.
A.10.9 Alternative analysis methods

A.10.9.1 Ultimate strength analysis
No guidance offered.

A.10.9.2 Types of analysis
When using the provisions of subclause 7.10 of ISO 19902 (reserve strength analysis), care should be taken in modelling nonlinear behaviour of chords and holding system of a jack-up structure.

A.11 Long-term applications

A.11.1 Applicability
No guidance offered.

A.11.2 Assessment data

A.11.2.1 Jack-up data
For the assessment of the jack-up for a long-term application, 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.

Such modifications can 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
Joint probability and/or directional metocean data can be used to optimise the ULS and FLS assessment for the long-term application.

The data required for a fatigue analysis should include long-term wave distribution and wave scatter data, refer to A.6.4.2.10.

A.11.2.3 Geotechnical data
Effects of seabed scour, differential settlement, consolidation settlement, reservoir subsidence and 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 affect on geotechnical properties due to top-hole construction activities, marine growth, expected subsidence, 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 location 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 advantageous. 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, etc) and, where available, the co-ordinates of the previous locations so that metocean parameters can be developed for use in historical analysis;
- hull elevation and orientation;
- water depth;
- penetration; and
- 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 the fatigue assessment need not 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 SCF (Stress Concentration Factors) from literature. Alternatively, more detailed fine mesh finite element models can be used to determine the hotspot stress ranges (suitable methodologies are given in References [A.11.3-1] to [A.11.3-7]).
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 clay and A.9.3.4.4 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 nonlinearities and dynamics:
The structural response of a jack-up is such that pure linear techniques can be inadequate. Therefore the analysis should include the nonlinear effects of the structure. These could include:

- Hydrodynamic actions,
- Large displacement effects (see 8.8.6),
- Dynamic amplification (see 10.5.2, 10.5.3),
- Leg-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 nonlinearity 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 the table below. These should be used accounting for 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 -2SD.

<table>
<thead>
<tr>
<th>Organisation</th>
<th>Document</th>
<th>Reference</th>
</tr>
</thead>
<tbody>
<tr>
<td>DNV</td>
<td>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.</td>
<td>[A.11.3-1] [A.11.3-2]</td>
</tr>
<tr>
<td>API</td>
<td>Methods are given in RP2A-LRFD-2003</td>
<td>[A.11.3-5]</td>
</tr>
<tr>
<td>UK HSE</td>
<td>Guidance is given in OTO 2001/015 and OTH92 390</td>
<td>[A.11.3-6] [A.11.3-7]</td>
</tr>
</tbody>
</table>
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 follows:

\[ f_{FD,e} \cdot D_e + f_{FD,s} \cdot D_s < 1.0 \quad (A.11.3-1) \]

where

- \( D_e \) = calculated existing fatigue damage prior to arriving at location
- \( D_s \) = calculated fatigue damage during planned operations on location
- \( f_{FD,s} \) = fatigue design factor, this is generally determined from the applicable table below
- \( f_{FD,e} \) = \( f_{FD,s} \), but not larger than 2, provided the detail has been inspected thoroughly before the long term application

**Table A.11.3-2a — Fatigue design factor \( f_{FD,s} \)**

<table>
<thead>
<tr>
<th>Fatigue Design Factor ((f_{FD,s}))</th>
<th>Full access for inspection and repair</th>
<th>Access for inspection, no repair during operation</th>
<th>No access for inspection, no repair during operation</th>
</tr>
</thead>
<tbody>
<tr>
<td>Full Redundancy/Minor Consequence</td>
<td>2</td>
<td>3</td>
<td>5</td>
</tr>
<tr>
<td>No Redundancy/Major Consequence</td>
<td>3</td>
<td>5</td>
<td>10</td>
</tr>
</tbody>
</table>

The values in the following table can be used for structures that are fully redundant i.e. the structure does not have single members or member connections that, when damaged, would cause a failure with major consequence. This is typical of RCS approved jack-ups with braced legs.

**Table A.11.3-2b — Fatigue design factor \( f_{FD,s} \) - redundant structure**

<table>
<thead>
<tr>
<th>Description (all structure is considered fully redundant)</th>
<th>Fatigue Design Factor ((f_{FD,s}))</th>
</tr>
</thead>
<tbody>
<tr>
<td>Hull structure</td>
<td></td>
</tr>
<tr>
<td>Primary hull structure</td>
<td>1</td>
</tr>
<tr>
<td>Leg to hull interface structure with access for inspection</td>
<td>2</td>
</tr>
<tr>
<td>Leg to hull interface structure without access for inspection</td>
<td>3</td>
</tr>
<tr>
<td>Leg structure in air</td>
<td></td>
</tr>
<tr>
<td>Leg chords, brace to chord joints, brace joints</td>
<td>2</td>
</tr>
<tr>
<td>Leg structure in splash zone</td>
<td></td>
</tr>
<tr>
<td>Leg chords, brace to chord joints, brace joints</td>
<td>3</td>
</tr>
<tr>
<td>Leg structure under water</td>
<td></td>
</tr>
</tbody>
</table>
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 compliance 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 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 compliance with the assessment assumptions.

A.11.3.5 Foundations

Settlements can occur as discussed in 11.3.5 and A.11.2.3, resulting in the loss of air gap or hull 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 over the long-term operation can result in changes in the foundation strength and stiffness. This affects the redistribution of the 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.

<table>
<thead>
<tr>
<th>Description (all structure is considered fully redundant)</th>
<th>Fatigue Design Factor ((F_{\text{FD}}))</th>
</tr>
</thead>
<tbody>
<tr>
<td>Leg chords, brace to chord joints, brace joints, leg to spudcan connection</td>
<td>3</td>
</tr>
<tr>
<td><strong>Leg structure under sea floor</strong></td>
<td></td>
</tr>
<tr>
<td>Leg chords, brace to chord joints, brace joints, leg to spudcan connection</td>
<td>5</td>
</tr>
<tr>
<td><strong>Spudcan</strong></td>
<td></td>
</tr>
<tr>
<td>Structure with access for inspection</td>
<td>3</td>
</tr>
<tr>
<td>Structure without access for inspection</td>
<td>5</td>
</tr>
</tbody>
</table>
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 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 (PSIIP)

The existing in-service inspection programme normally required by the RCS should be modified and updated to reflect the requirements for the planned long-term application defined in A.11.4.1.

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 value of the PSIIP and for extending the jack-up’s time on location beyond that originally planned.

A.11.4.3 Alternative project specific in-service inspection programme

An alternative PSIIP 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 Structural strength

A.12.1 Applicability

A.12.1.1 General

This clause applies to steel structures only. Where necessary, the equations included in this clause 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 structure as 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 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 prismatic structural member is usually comprised of several structural components. Table A.12.2-1 shows example components for typical jack-up chords comprising split-tubulars, rack plates, side plates and back plates. A component is 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. Prismatic members should be assessed using the provisions of A.12.6.
Tubulars should be assessed as structural members using the provisions of A.12.5.

In this clause, 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 yield strength in stress units.

**A.12.1.2 Truss type legs**

No guidance offered.

**A.12.1.3 Other leg types**

No guidance offered.

**A.12.1.4 Fixation system and/or elevating system**

No guidance offered.

**A.12.1.5 Spudcan strength including connection to the leg**

No guidance offered.

**A.12.1.6 Overview of the assessment procedure**

No guidance offered.

**A.12.2 Classification of member cross-sections**

**A.12.2.1 Member type**

No guidance offered.

**A.12.2.2 Material yield strength**

The value of yield strength taken from a tensile test should correspond to the 0.2% offset value. Where this is greater than 90% of the ultimate tensile strength (UTS), the yield strength used in this clause should be taken as 90% of UTS. The following variables are used in this clause:

\[
F_y = \text{yield strength in stress units} \\
= \text{minimum of the yield strength and 90\% of the UTS} \\
F_{yi} = \text{yield strength of the } i^{th} \text{ component of a prismatic member cross-section, in stress units} \\
= \text{minimum of the yield strength and 90\% of the UTS of the } i^{th} \text{ component of the cross-section} \\
F_{ymin} = \text{minimum yield strength, } F_{yi}, \text{ of all components in a prismatic member cross-section, in stress units} \\
F_{yeff} = \text{effective yield strength of a prismatic member cross-section, in stress units, determined from the plastic tensile axial strength divided by the minimum cross-sectional area.}
\]

**A.12.2.3 Classification definitions**

**A.12.2.3.1 Tubular member classification**

A tubular section is a class 1 section when:
\[ D/t \leq 0,0517 \frac{E}{F_y} \]  
\((A.12.2-1)\)

where

\[ D = \text{outside diameter} \]
\[ t = \text{wall thickness} \]
\[ F_y = \text{yield strength in stress units} \]
\[ E = \text{elastic modulus} \]

NOTE Compliance with class 1 classification is only relevant 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 Prismatic member classification

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 4. The limits given in Table A.12.2-1 tend to be conservative as in most cases there is additional support of 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 may be possible to justify the use of alternative limits through rational analysis.

**Table A.12.2-1 Classification limits for prismatic members containing curved components**

<table>
<thead>
<tr>
<th>Class</th>
<th>( D/t ) limits</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Section in bending</td>
</tr>
<tr>
<td>1</td>
<td>( D/t \leq 0,052 \frac{E}{F_y} )</td>
</tr>
<tr>
<td>2</td>
<td>( D/t \leq 0,103 \frac{E}{F_y} )</td>
</tr>
<tr>
<td>3</td>
<td>( D/t \leq 0,220 \frac{E}{F_y} )</td>
</tr>
<tr>
<td>4</td>
<td>( D/t &gt; 0,220 \frac{E}{F_y} )</td>
</tr>
</tbody>
</table>

When classifying prismatic components in accordance with Table A.12.2-2 to 4, a distinction is made between internal components and outstand components as follows:

— Internal components: 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

— Outstand components: 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. Slender components within a cross-section may be ignored provided that the remaining cross-section is used for all aspects of the assessment. However, if a slender 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 Table A.12.2-2 to 4, the dimensions to be used are those given in the 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 which do not satisfy the applicable following simplified lateral torsional buckling (LTB) criteria should be assessed further to determine a reduced representative member bending strength, \( M_b \), using the guidance in A.12.6.2.6.

Singly symmetric open sections, from F2-5 of AISC\(^{[A.12.5-1]}\):

\[
\frac{L_b}{r_z} \leq 1.76 \sqrt{\frac{E}{F_{y,ltb}}} 
\]  
(A.12.2-2a)

or for any closed section, derived from BS 5400-3\(^{[A.12.5-12]}\):

\[
\frac{L_b}{r_z} \leq \frac{0.36 I_z E}{Z_p F_{y, min}} \left( \frac{AJ}{(I_y - I_z)(I_y - J / 2.6)} \right) 
\]  
(A.12.2-2b)

where

\[
I_y = \text{major axis second moment of area of the gross cross-section} \\
I_z = \text{minor axis second moment of area of the gross cross-section} \\
L_b = \text{effective length of beam-column between supports} \\
A = \text{gross cross-sectional area} \\
J = \text{torsion constant} = \frac{4 A_o^2}{\Sigma (b/t)} \\
\]

in which

\[
A_o = \text{the area enclosed by the median line of the perimeter material of the section} \\
b, t = \text{the width and thickness, respectively, of each component (wall of the section) forming the closed perimeter.} \\
L_b = \text{laterally unbraced length; i.e. length between points which are either braced against lateral displacement of the compression flange or braced against twist of the cross section in addition to lateral support.} \\
r_z = \text{radius of gyration about the minor axis as defined in equation A.12.3-7.} \\
F_{y,ltb} = \text{yield strength, } F_y \text{ of the material that first yields when bending about the minor axis.} \\
\text{Conservatively, } F_y \text{ may be taken as the maximum yield strength of all the components in a prismatic cross section} \\
E = \text{Young's modulus} \\
Z_p = \text{fully plastic effective section modulus about the major axis determined from equation A.12.3-2} \\
F_{y,min} = \text{minimum yield strength of the cross section as defined in A.12.2.2} \]

A.12.2.3.3 Reinforced components
Reinforcement of member cross-sections is often of the form shown in Figure A.12.2-1.

![Figure A.12.2-1 Definitions for reinforced plate](image)

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:

1) the reinforcing plate (using \( t_2 \)) over the width \( b_2 \), using increased buckling coefficient (see below)
2) the combined plate using \( t_{\text{check}} \) over width \( b_1 \)
3) the base plate (using \( t_1 \)) over the width \( b_2 \) using increased buckling coefficient (see below)
4) the base plate (using \( t_1 \)) over the dimension of the unreinforced widths (conservatively taken as \( b_1 - b_2 \))

If the cross section is found to be slender (class 4), then the effective width of each of the base plate, reinforcing plate, and the combined plate should be determined from Table A.12.3-2.

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 1 and 3 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 Table A.12.2-2 and 4 depending on the type of in-plane loading. The value of yield stress to be 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 4 depending on the type of in-plane loading and support conditions. The value of thickness \( t_{\text{check}} \) to be used with width \( b_1 \) in the formulae in Table A.12.2-2 and A.12.2-4 should be

\[
t_{\text{check}} = \left(t_{\text{eff}} t_1\right)^{1/4}
\]

where

\[
t_{\text{eff}} = \left(12 I / b_1^3\right)^{1/3}
\]

\[
I = \left[b_1 (t_1 + t_2)^3 - (b_1 - b_2) t_2^3\right] / 3 - A(t_1 + t_2 - y_1)^2
\]
ISO/DIS 19905-1

\[ y_1 = \frac{b_1 t_1^2 + b_2 t_2 (2t_1 + t_2)}{(2A)} \]  
\[ A = b_1 t_1 + b_2 t_2 \]  

(A.12.2-6)  

(A.12.2-7)

The value of yield stress to be used in Table A.12.2-2 to 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 limiting components

<table>
<thead>
<tr>
<th>Class</th>
<th>Type</th>
<th>Section in bending</th>
<th>Section in compression</th>
</tr>
</thead>
<tbody>
<tr>
<td>Plastic - Class 1</td>
<td>Rolled or Welded</td>
<td>( b/t_1 \leq 1.03\sqrt{(E/F_y)} )</td>
<td>( b/t_1 \leq 1.03\sqrt{(E/F_y)} )</td>
</tr>
<tr>
<td>Compact - Class 2</td>
<td>Rolled or Welded</td>
<td>( b/t_1 \leq 1.17\sqrt{(E/F_y)} )</td>
<td>( b/t_1 \leq 1.17\sqrt{(E/F_y)} )</td>
</tr>
<tr>
<td>Elastic stress distribution in component and across section (compression positive)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Semi-Compact - Class 3</td>
<td>Rolled or Welded</td>
<td>( b/t_1 \leq 1.44\sqrt{(E/F_y)} )</td>
<td>( b/t_1 \leq 1.44\sqrt{(E/F_y)} )</td>
</tr>
<tr>
<td>Slender - Class 4</td>
<td>Rolled or Welded</td>
<td>( b/t_1 &gt; 1.44\sqrt{(E/F_y)} )</td>
<td>( b/t_1 &gt; 1.44\sqrt{(E/F_y)} )</td>
</tr>
</tbody>
</table>
Table A.12.2-3 - Cross-section classification - flange outstand components

<table>
<thead>
<tr>
<th>Class</th>
<th>Type</th>
<th>Flange subject to compression</th>
<th>Flange subject to compression and bending</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>Tip in compression</td>
</tr>
<tr>
<td>Plastic -</td>
<td>Rolled</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Class 1</td>
<td>Welded</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Maximum compression at tip</td>
</tr>
<tr>
<td>Plastic -</td>
<td>Rolled</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Class 2</td>
<td>Welded</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Semi-Compact</td>
<td>Rolled</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Class 3</td>
<td>Welded</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Slender</td>
<td>Rolled or</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Class 4</td>
<td>Welded</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

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.
### Table A.12.2-4 - Cross-section classification - web internal components

<table>
<thead>
<tr>
<th>Class</th>
<th>Web subject to Bending</th>
<th>Web subject to Compression</th>
<th>Web subject to bending and compression</th>
</tr>
</thead>
</table>
| Plastic stress distribution in component (compression positive) | $\alpha = 0.5$ | $\alpha = 1.0$ | when $\alpha > 0.5$  
$\frac{dt_\text{w}}{t_\text{w}} \leq 2.56 \sqrt{\frac{E}{F_\text{y}}} \left(\frac{1}{\alpha}\right)$ |
| Plastic - Class 1 |  
$\frac{dt_\text{w}}{t_\text{w}} \leq 2.56 \sqrt{\frac{E}{F_\text{y}}}$ |  
$\frac{dt_\text{w}}{t_\text{w}} \leq 1.03 \sqrt{\frac{E}{F_\text{y}}}$ |  
when $\alpha > 0.5$  
$\frac{dt_\text{w}}{t_\text{w}} \leq 5.18 \sqrt{\frac{E}{F_\text{y}}} \left(6.043\alpha-1\right)$ |
| Compact - Class 2 |  
$\frac{dt_\text{w}}{t_\text{w}} \leq 3.09 \sqrt{\frac{E}{F_\text{y}}}$ |  
$\frac{dt_\text{w}}{t_\text{w}} \leq 1.17 \sqrt{\frac{E}{F_\text{y}}}$ |  
when $\alpha > 0.5$  
$\frac{dt_\text{w}}{t_\text{w}} \leq 4.82 \sqrt{\frac{E}{F_\text{y}}} \left(5.12\alpha-1\right)$ |
| Elastic stress distribution in component (compression positive) |  |  | when $\alpha \leq 0.5$  
$\frac{dt_\text{w}}{t_\text{w}} \leq 1.28 \sqrt{\frac{E}{F_\text{y}}} / \alpha$ |
| Semi-Compact - Class 3 |  
$\frac{dt_\text{w}}{t_\text{w}} \leq 4.14 \sqrt{\frac{E}{F_\text{y}}}$ |  
$\frac{dt_\text{w}}{t_\text{w}} \leq 1.44 \sqrt{\frac{E}{F_\text{y}}} \left(0.674+0.327\psi\right)$ |  
when $\psi > -1.0$  
$\frac{dt_\text{w}}{t_\text{w}} \leq 1.44 \sqrt{\frac{E}{F_\text{y}}} \left(0.674+0.327\psi\right)$ |
| Slender - Class 4 |  
$\frac{dt_\text{w}}{t_\text{w}} >$ than for Class 3 |  
$\frac{dt_\text{w}}{t_\text{w}} >$ than for Class 3 |  
$\frac{dt_\text{w}}{t_\text{w}} >$ than for Class 3 |
A.12.3 Section properties

A.12.3.1 General

Cross-sectional properties appropriate for the strength assessment of prismatic members of all classes should be determined as described in A.12.3.2 to A.12.3.4, and 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 2 sections in place of plastic section properties (e.g. for Euler amplification calculations or structural analysis input 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 prismatic section.

Cross-sectional properties for tubular members are specified in A.12.5.

The cross-section 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 approximately 10% of the maximum rack tooth area when determining the leg stiffness, however, this additional material should not normally be included when calculating the section properties for strength assessment. However, these increased cross-section properties may be used when determining the radii of gyration used in the determination of the column buckling strength (A.12.6.2.4) and moment amplification (A.12.4). This increase in cross-section should not be used elsewhere in the strength calculations.

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 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 may be used to determine the relevant properties.

Note to reviewers: It is intended that the current approach using the "effective properties" $A_p$ and $Z_p$ together with the minimum yield strength $F_{ymin}$ be replaced by an approach that uses the actual properties and the effective yield strength $F_{yeff}$, so we will define that the nominal strength $= A_p F_{yeff}$ here where

$$F_{yeff} = (\Sigma F_{yi}A_i)/\Sigma A_i,$$

with $F_{yeff} = \text{The effective yield strength in stress units to be used with the area } \Sigma A_i.$

This is preferred to minimise the potential for confusion as we already have weight, buoyancy and stiffness areas. It should however be noted that this change will have no technical impact.

For axial tension and compression, the fully plastic effective cross-sectional area for use in a strength assessment, $A_p$ is:

$$A_p = (\Sigma F_{yi}A_i)/F_{ymin} \quad (A.12.3-1)$$

where
\( F_{yi} = \) yield strength of the \( i^{th} \) component comprising the structural member, as defined in A.12.2.2.

\( A_i = \) cross-sectional area of the \( i^{th} \) component comprising the structural member

\( F_{ymin} = \) yield strength to be used in the calculation, 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:

\[
Z_p = \frac{\sum F_{yi} d_i A_i}{F_{ymin}} \tag{A.12.3-2}
\]

where

\( d_i = \) distance between the centroid of the \( i^{th} \) 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 to 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, \( A_f \), the cross-sectional area, \( I_f \), the second moment of area and \( S_f \), the elastic section modulus should be based on elastic properties assuming that the full cross-section is effective.

\[
A_f = \sum A_i \tag{A.12.3-3}
\]

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.12.2-2 to 4. Cross-section 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 in this clause.

When analysing structures that contain class 4 sections, care should be taken when determining the force distributions. It is generally recommended that the structural analysis is performed using full elastic section properties and the reduced section properties are only used 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 (where compression is taken to be positive and tension to be negative):

\[
\begin{align*}
\psi &= \text{compression to bending stress ratio} \\
\sigma_1 &= \text{compressive stress if } \sigma_2 \text{ tensile or larger compressive stress if } \sigma_2 \text{ compressive} \\
\sigma_2 &= \text{tensile stress if } \sigma_2 \text{ tensile or smaller compressive stress if } \sigma_2 \text{ compressive} \\
k &= \text{buckling coefficient} \\
\rho &= \text{reduction coefficient} \\
\lambda_p &= \text{plate slenderness parameter} \\
\lambda_{plim} &= \text{limiting plate slenderness ratio} \\
\lambda_{po} &= \text{plate slenderness ratio coefficient}
\end{align*}
\]

When determining effective widths for web internal components, the stress ratio \( \psi \) 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 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 \( A_{\text{eff},i} \) of a compressed component should be found as the product of its thickness and its effective width. The total effective area, \( A_{\text{ec}} \), is the sum of the component effective areas:

\[
A_{\text{ec}} = \sum A_{\text{eff},i} \tag{A.12.3-4}
\]

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 should be determined from Table A.12.3-1 (b).

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, effective areas as defined in A.12.3.4.2 for components subject to compression, and effective areas as defined immediately above 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 section moduli $S_e$ used in the calculations should encompass all critical points on the cross-section.

$$S_e = \frac{I_e}{y_i} \quad (A.12.3-5)$$

where

$y_i = \text{The distance from the neutral axis associated with } I_e \text{ to the critical point } i$

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 Slender Sections

<table>
<thead>
<tr>
<th>(a) Compression flange internal components</th>
<th>(b) Outstand components under compression and/or bending</th>
</tr>
</thead>
<tbody>
<tr>
<td>$0.72 t f \sqrt{E/F_y}$</td>
<td>$0.50 t f \sqrt{E/F_y}$</td>
</tr>
<tr>
<td>$0.75 t f \sqrt{(k_\sigma E/F_y)}$</td>
<td>$0.75 t f \sqrt{(k_\sigma E/F_y)}$</td>
</tr>
</tbody>
</table>

See Table A.12.2-3 for definition of $k_\sigma$.
A.12.3.5 Cross section properties for assessment

A.12.3.5.1 Tension

In tension, the cross-sectional area to be used in assessment should be $A_t$ where

$$ A_t = A_p \quad \text{for class 1 plastic or class 2 compact sections (see equation A.12.3-1)} $$

$$ A_t = A_l \quad \text{for class 3 semi-compact as defined in equation A.12.3-3.} $$

$$ A_t = A_l \quad \text{as defined in equation A.12.3-3 for class 4 slender sections in tension across the whole of the cross-section (including bending); otherwise use } A_{ec} \text{ for class 4 sections as defined by equation A.12.3-4.} $$

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 to be used in assessment should be $A_c$, where

- $A_c = A_p$ for class 1 plastic or class 2 compact sections (see equation A.12.3-1)
- $A_c = A_f$ for class 3 semi-compact sections as defined in equation A.12.3-3
- $A_c = A_{ec}$ for class 4 slender sections as defined in equation A.12.3-4.

A.12.3.5.3 Flexure

In flexure, the second moment of area with respect to major ($y$) and minor ($z$) axis bending to be used in assessment should be determined from:

- $I_y, I_z = I_1$ for class 1 plastic and class 2 compact sections as defined in A.12.3.2.2
- $I_y, I_z = I_1$ for class 3 semi-compact sections as defined in A.12.3.3
- $I_y, I_z = I_e$ 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:

- $S_y, S_z = Z_p$ for Class 1 Plastic or Class 2 Compact sections (see Equation A.12.3-2)
- $S_y, S_z = S_f$ for Class 3 Semi-compact sections as defined in A.12.3.3 for each critical stress location.
- $S_y, S_z = S_e$ 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 to be used for lateral-torsional buckling considerations, $r_z$, should be determined from:

- $r_z = (I_z/A_c)^{0.5}$ for sections in classes 1 to 3
- $r_z = (I_e/A_{ec})^{0.5}$ for sections in class 4

The cross-section properties used when determining the radius of gyration for buckling strength of chords may include approximately 10% of the maximum rack tooth area. This increase in the cross-section should not be used elsewhere in the strength calculations (see A.12.3.1).

A.12.4 Effects of axial force on bending moment

A.12.4.1 General

Member Euler amplification ($p$-$\delta$) applies to all members in axial compression.

For class 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 member Euler 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' as given in Equation A.12.4-1. There is no eccentricity for tubular members or for prismatic members with material symmetry.

The corrected effective moment, $M_{ue}$, should be calculated for each axis of bending from:

$$M_{ue} = M_u + eP_u \quad (A.12.4-1)$$

where

- $M_u =$ moment in a member determined in an analysis which includes global P-∆ effects.
- $P_u =$ the axial force in the member, determined in an analysis which includes global P-∆/hull-sway effects.
- $e =$ the eccentricity between the axis used for structural analysis and that used for structural strength checks, taking due account of sign in combination with the sign convention for $P_u$.

- for class 1 and 2 members, 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;
- for class 3 members, $e_a$ as defined in A.12.6.2.3;
- for class 4 members, the distance between the neutral axes of the full and effective cross sections, orthogonal to the axis of bending under consideration;
- $= 0$ if the structural model fully accounts for the offset between the neutral axes of the modelled member in the strength checks.
- $= 0$ for tubular members; for other cross-sections in class 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 from:

$$M_{ua} = B M_{ue} \quad (A.12.4-2)$$

where

- $M_{ue}$ is as defined in A.12.4.2
- $B =$ member moment amplification factor for the axis under consideration;
- $B = 1.0$ (i) for members in tension or
- (ii) for 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 $M_u$.

$$B = \frac{C_m}{(1 - P_u / P_e)}$$

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

$$B = \frac{C_m}{(1 - P_u / P_e)} \geq 1.0$$

as above, but where the interaction surface approach of A.12.6.3.3 is to be applied

where

$$P_E = \left(\pi^2 A_c E / (K L/r)^2\right)$$

and is to be calculated for the plane of bending.

$A_c$ is defined in A.12.3.5.2

$K$ and $C_m$ are given in Table A.12.4-1

$r = \text{radius of gyration for the plane of flexural buckling}$

$K = \text{effective length factor for the plane of flexural buckling}$

$L = \text{unbraced length of member for plane of flexural buckling normally taken as:}$

- face to face for braces
- braced point to braced point 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, $B$ 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

<table>
<thead>
<tr>
<th>Structural member</th>
<th>( K )</th>
<th>( C_m^{(1)} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Tubular or box complete legs</td>
<td>2,0(^{(2)})</td>
<td>(a)</td>
</tr>
<tr>
<td>Chords with lateral loading</td>
<td>1,0</td>
<td>(c)</td>
</tr>
<tr>
<td>Chords without lateral loading</td>
<td>1,0</td>
<td>(b)</td>
</tr>
<tr>
<td>Tubular braces</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Primary diagonals and horizontals</td>
<td>0,7</td>
<td>(b) or (c)</td>
</tr>
<tr>
<td>K-braces(^{(3)})</td>
<td>0,7</td>
<td>(c)</td>
</tr>
<tr>
<td>X-brace(^{(3)})</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Longer segment length</td>
<td>0,8</td>
<td>(c)</td>
</tr>
<tr>
<td>Full length(^{(4)})</td>
<td>0,7</td>
<td>(c)</td>
</tr>
<tr>
<td>Secondary horizontals</td>
<td>0,7</td>
<td>(c)</td>
</tr>
</tbody>
</table>

1. The value of \( C_m \) 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 \( C_m = 0,85 \)
   (b) For members whose ends are unrestrained against sidesway \( C_m = 1,0 \)
   (c) For members with no transverse loading, ignoring self-weight:

\[
C_m = 0,6 - 0,4 \frac{M_1}{M_2}
\]

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 \( M_2 \)

2. Alternatively use effective length alignment chart in Figure A.12.4-1.

3. 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.

4. 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

$$G = \frac{I_c}{\sum I_0}$$

in which $\Sigma$ 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.

**A.12.5 Strength of tubular members**

**A.12.5.1 Applicability**

The strength of unstiffened tubular members that satisfy the following condition should be assessed in accordance with this clause.

Any tubular with $D/t < 120$  \hspace{1cm} (A.12.5-1)

Tubulars that do not satisfy this condition should be assessed using alternative methods that result in levels of reliability comparable to those implicit in this document, such as References [A.12.5-3] and [A.12.5-4].

The formulations in this clause, based on the $D/t$ limit above, 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 a prismatic member unless it is equipped with adequately sized reinforcement. 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 the tubular formulations may be used.

The formulations are considered applicable for steels with yield strength of up to 700 N/mm$^2$. The yield strength used in this clause should be as specified in A.12.2.2.
NOTE The formulations for tubular members are based on ISO 19902 Clause 13. However, for use in this document, the ISO 19902 formulations have been converted to a force base rather than a stress base.

The provisions given in this clause ignore the effect of hydrostatic pressure. The condition under which hydrostatic pressure may be ignored for a specific member is given by:

\[
(D/t)_m = \frac{211}{d^{0.335}}
\]

where

\[
d = \text{effective head of water applicable to the tubular in question}
\]

\[
= \text{depth below the water surface (including penetration into the seabed) + } p \cdot \gamma' / \rho_w
\]

\[
p = \text{depth of penetration below sea floor (zero if above sea floor)}
\]

\[
\gamma' = \text{submerged unit weight of the soil}
\]

\[
\rho_w = \text{weight density of water}
\]

\[
(D/t)_m = \text{maximum } D/t \text{ ratio possible given } d.
\]

For convenience, some typical \((D/t)_m\) values are listed in Table A.12.5-1.

Table A.12.5-1 Maximum \((D/t)_m\) ratios for given depth

<table>
<thead>
<tr>
<th>Water depth d (m)</th>
<th>Maximum tubular ((D/t)_m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>43</td>
<td>60,0</td>
</tr>
<tr>
<td>50</td>
<td>56,9</td>
</tr>
<tr>
<td>75</td>
<td>49,7</td>
</tr>
<tr>
<td>100</td>
<td>45,1</td>
</tr>
<tr>
<td>125</td>
<td>41,9</td>
</tr>
<tr>
<td>150</td>
<td>39,4</td>
</tr>
<tr>
<td>200</td>
<td>35,8</td>
</tr>
</tbody>
</table>

If the member \(D/t\) exceeds the limiting value \((D/t)_m\) for the depth of the tubular, 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 Axial tensile strength check

Tubular members subjected to axial tensile forces, \(P_{ut}\), should satisfy:

\[
P_{ut} \leq A \cdot F_y / \gamma_{R,Ti}
\]

where

\[
F_y = \text{yield stress as defined in A.12.2.2}
\]

\[
A = \text{total cross-sectional area}
\]
ISO/DIS 19905-1

\[ \gamma_{R,T} = \text{partial resistance factor for axial tension, 1.05} \]

### A.12.5.2.2 Axial compressive strength check

Tubular members subjected to axial compressive forces, \( P_{uc} \), should satisfy:

\[ P_{uc} \leq \frac{P_a}{\gamma_{R,Tc}} \]  \hspace{1cm} (12.5-4)

where

\[ P_a = \text{representative compressive strength as determined in A.12.5.2.3} \]

\[ \gamma_{R,Tc} = \text{partial resistance factor for axial compressive strength, 1.15} \]

### A.12.5.2.3 Local buckling strength

The representative local buckling strength, \( P_{yc} \), should be determined from:

\[
P_{yc} = A F_y \quad \text{for} \quad \frac{A F_y}{P_{xe}} \leq 0.170 \quad \text{(A.12.5-5a)}
\]

\[
= \left[ 1.047 - 0.274 \frac{A F_y}{P_{xe}} \right] A F_y \quad \text{for} \quad 0.170 < \frac{A F_y}{P_{xe}} \leq 200 \frac{F_y}{E} \quad \text{(A.12.5-5b)}
\]

where

\[ F_y = \text{yield stress as defined in A.12.2.2} \]

\[ A = \text{total cross-sectional area} \]

\[ P_{xe} = \text{representative elastic local buckling strength} \]

\[ = 2 C_x E A \left( \frac{t}{D} \right) \]

\[ C_x = \text{critical elastic buckling coefficient} \]

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 \( \frac{A F_y}{P_{xe}} \) given in Equations 12.5-5.

### A.12.5.2.4 Column buckling strength

The representative axial compressive strength of tubular members, \( P_a \), should be determined from:

\[
P_a = \left[ 1.0 - 0.278 \lambda^2 \right] P_{yc} \quad \text{for} \quad \lambda \leq 1.34 \quad \text{(A.12.5-6a)}
\]

\[
= 0.9 \frac{P_{yc}}{\lambda^2} \quad \text{for} \quad \lambda > 1.34 \quad \text{(A.12.5-6b)}
\]

\[ \lambda = \left[ \frac{P_{yc}}{P_E} \right]^{0.5} \quad \text{(A.12.5-7)}
\]

where

\[ P_{yc} = \text{representative local buckling strength (see A.12.5.2.3)} \]

\[ \lambda = \text{column slenderness parameter} \]

\[ P_E = \text{smaller of the Euler buckling strengths about the y or z direction} \]
A.12.5.2.5 Bending strength check

Tubular members subjected to bending moments, \( M_u \), should satisfy:

\[
M_u \leq \frac{M_b}{\gamma_{R,Tb}}
\]  
(A.12.5-8)

where

\[
M_u = M_{uy} \text{ or } M_{uz}
\]

the bending moment about member y- and z-axes respectively due to factored actions

\[
M_b = \text{representative bending moment strength, determined from:}
\]

\[
M_b = M_p
\]

for \( (F_y D)/(E t) \leq 0,0517 \)  
(A.12.5-9a)

\[
= [1,13 - 2,58 (F_y D)/(E t)] M_p
\]

for \( 0,0517 < (F_y D)/(E t) \leq 0,1034 \)  
(A.12.5-9b)

\[
= [0,94 - 0,76 (F_y D)/(E t)] M_p
\]

for \( 0,1034 < (F_y D)/(E t) \leq 120 (F_y / E) \)  
(A.12.5-9c)

\[
M_p = \text{plastic moment strength}
\]

\[
= F_y [D^3 - (D - 2t)^3] / 6
\]

\[
\gamma_{R,Tb} = \text{partial resistance factor for bending, 1.05}
\]

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 forces should satisfy the following condition along their length:

\[
\gamma_{R,Tt} \frac{P_{ut}}{(A F_y)} + \gamma_{R,Tb} (M_{uy}^2 + M_{uz}^2)^{0.5} / M_b \leq 1.0
\]  
(A.12.5-10)

where

\[
P_{ut} = \text{axial tensile force due to factored actions}
\]

\[
A = \text{total cross-sectional area}
\]

\[
F_y = \text{yield stress as defined in A.12.2.2}
\]

\[
M_{uy}, M_{uz} = \text{bending moments about member y- and z-axes respectively due to factored actions determined in an analysis which includes global P-\Delta effects}
\]

\[
M_b = \text{representative moment strength, as defined in equations A.12.5-9}
\]
\[ \gamma_{R,T1} = \text{partial resistance factor for axial tension, } 1,05 \]
\[ \gamma_{R,Tb} = \text{partial resistance factor for bending, } 1,05 \]

### A.12.5.3.2 Axial compression and bending strength check

Tubular members subjected to combined axial compression and bending forces should satisfy the following conditions at all cross sections along their length:

**Beam-column check:**

\[
(\gamma_{R,Tc} P_{uc}/P_a) + \left( \gamma_{R,Tb}/M_b \right) \left( M_{uy}^2 + M_{uz}^2 \right)^{0.5} \leq 1.0 \tag{A.12.5-11}
\]

**Local strength check:**

\[
(\gamma_{R,Tc} P_{uc}/P_y) + \left( \gamma_{R,Tb}/M_b \right) \left( M_{uy}^2 + M_{uz}^2 \right)^{0.5} \leq 1.0 \tag{A.12.5-12}
\]

where

- \( P_{uc} \) = axial compressive force due to factored actions
- \( P_y \) = the representative local buckling strength in A.12.5.2.3,
- \( P_a \) = as defined in A.12.5.2.4
- \( M_{uy} \) = corrected effective bending moment about member y-axis due to factored actions determined in an analysis which includes global P-\( \Delta \) effects
- \( M_{uz} \) = corrected effective bending moment about member z-axis due to factored actions determined in an analysis which includes global P-\( \Delta \) effects
- \( M_{uy} \) = amplified bending moment about member y-axis due to factored actions from A.12.4.3
- \( M_{uz} \) = amplified bending moment about member z-axis due to factored actions from A.12.4.3
- \( M_b \) = representative bending strength, as defined in equations A.12.5-9
- \( \gamma_{R,Tb} \) = partial resistance factor for bending, 1.05
- \( \gamma_{R,Tc} \) = partial resistance factor for axial compressive strength, 1.15

### A.12.5.3.3 Beam shear strength check

Tubular members subjected to beam shear forces should satisfy:

\[
V \leq P_v/\gamma_{R,TV} \tag{A.12.5-13}
\]

where

- \( V \) = beam shear due to factored actions
- \( P_v \) = representative shear strength
  \[ = A F_v/(2\sqrt{3}) \]
- \( A \) = total cross-sectional area
- \( \gamma_{R,TV} \) = partial resistance factor for beam shear strength, 1.05
A.12.5.3.4 Torsional shear strength check

Tubular members subjected to torsional shear forces should satisfy:

\[ T_u \leq \frac{T_v}{\gamma_{R,Tv}} \]  \hspace{1cm} (A.12.5-14)

where

- \( T_u \) = torsional moment due to factored actions
- \( T_v \) = representative torsional strength
  \[ T_v = 2 I_p F_y / (D \sqrt{3}) \]
- \( I_p \) = polar moment of inertia
  \[ I_p = (\pi / 32) [D^4 - (D - 2t)^4] \]

A.12.6 Strength of prismatic members

A.12.6.1 General

The structural strength provisions for rolled and welded prismatic members are generally based on AISC 2005 Specification for Structural Steel Buildings, Reference [A.12.5-1]. The AISC 2005 specification for LRFD was interpreted and, in some cases, modified for use in the assessment of mobile jack-up structures. The formulations 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\(^\text{[A.12.6-5]}\). Interpretation of the specifications was necessary to enable a straight-forward method to be presented 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:

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 which has been identified in the standard AISC interaction equations.

The yield strength used in this subclause should be as specified in A.12.2.2.

The resistance factors used in the AISC 2005 LRFD specification have been adopted.

The effects of hydrostatic loading on prismatic members should be considered. The critical condition for hydrostatic loading on prismatic chord members is likely to occur when high spudcan fixity results in high chord axial loads in deepwater.

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 may be used to determine the effects of the stiffening provided by the non-tubular components.

NOTE << It is intended that "no need for separate consideration" water depth limits, similar to those given for tubulars in Table A.12.5-1, will be added for flat plates. These are to incorporate axial load and hydrostatic pressure for W/t up to 33 and \( F_y \) up to 700 MPa. These are expected from PAFA/UK HSE. >>

In the following sub-sections, \( y \) and \( z \) are used to define the major and minor axes of a prismatic member.
A.12.6.2 Prismatic members subjected to tension, compression, bending or shear

A.12.6.2.1 General

Prismatic members subjected to axial tension, axial compression, bending or shear should satisfy the applicable strength and stability formulations specified in this clause.

A.12.6.2.2 Axial tensile strength check

Prismatic members subjected to axial tensile forces, \( P_{\text{ut}} \), should satisfy:

\[
P_{\text{ut}} \leq \frac{P_t}{\gamma_{R,Pl}}
\]

where

\( P_t = \) axial tensile strength of prismatic members

\( = \sum(F_i A_i) \)

\( F_i = \) yield strength of the \( i^{\text{th}} \) component comprising the structural member, as defined in A.12.2.2;

\( A_i = \) cross-sectional area of the \( i^{\text{th}} \) component comprising the structural member;

\( \gamma_{R,Pl} = \) partial resistance factor for axial tension, 1.05.

A.12.6.2.3 Axial compressive local strength check

Prismatic members subjected to axial compressive forces, \( P_{\text{uc}} \), should satisfy:

\[
P_{\text{uc}} \leq \frac{P_{\text{pl}}}{\gamma_{R,Pl}}
\]

where

\( \gamma_{R,Pl} = \) partial resistance factor for local axial compressive strength, 1.1;

\( P_{\text{pl}} = \) the local compressive axial strength of prismatic members;

\( = \sum F_i A_i \) for class 1 and 2 prismatic members (A.12.6-3)

\( = \sum F_i A_i - (\sum F_i A_i - F_{\text{ymin}} \sum A_i) \left( \frac{\lambda - \lambda_p}{\lambda_c - \lambda_p} \right) h \) for class 3 prismatic members (A.12.6-4)

\( = F_{\text{ymin}} A_c \) for class 4 prismatic members; (A.12.6-5)

\( F_i = \) yield strength of the \( i^{\text{th}} \) component comprising the structural member, as defined in A.12.2.2;

\( F_{\text{ymin}} = \) minimum yield stress of the cross section as defined in A.12.2.2.

\( A_i = \) cross-sectional area of the \( i^{\text{th}} \) component comprising the structural member;

\( A_c = \) area of section from A.12.3.5.2;

\( h = \) subscript referring to the component that produces the smallest value of \( P_{\text{pl}} \).
\[ \lambda = \frac{h}{t} \text{ or } \frac{2R}{t} \text{ as applicable for component } h; \]

\( \lambda_p \) is determined for component \( h \) from: Tables A.12.2-2 to A.12.2-4, and are as below:

a) For rectangular rolled or welded flange components supported along both edges
\[ \lambda_p = 1,17 \sqrt{\left(\frac{E}{F_{yi}}\right)} \] (A.12.6-6a)

b) For rectangular rolled flange or web components supported along one edge
\[ \lambda_p = 0,37 \sqrt{\left(\frac{E}{F_{yi}}\right)} \] (A.12.6-6b)

c) For rectangular welded flange or web components supported along one edge
\[ \lambda_p = 0,33 \sqrt{\left(\frac{E}{F_{yi}}\right)} \] (A.12.6-6c)

d) For rectangular rolled or welded web components supported along both edges
\[ \lambda_p = \left[ 4,82 \sqrt{\left(\frac{E}{F_{yi}}\right)} \right] / (5,12 \alpha - 1) \quad \text{for } \alpha > 0,5 \] (A.12.6-6d)
\[ \lambda_p = \left[ 1,55 \sqrt{\left(\frac{E}{F_{yi}}\right)} \right] / \alpha \quad \text{for } \alpha \leq 0,5 \] (A.12.6-6e)

where \( \alpha \) is a factor that varies depending on the 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.

e) For components derived from circular tubes
\[ \lambda_p = 0,077 \frac{E}{F_{yi}} \] [Table A.12.2-1] (A.12.6-6f)

\( \lambda_r \) is determined for component \( h \) from Tables A.12.2-2 to A.12.2-4, and are as below:

a) For rectangular rolled or welded flange components supported along both edges
\[ \lambda_r = 1,44 \sqrt{\left(\frac{E}{F_{yi}}\right)} \] (A.12.6-7a)

b) For rectangular rolled flange or web components supported along one edge
\[ \lambda_r = 0,55 \sqrt{\left(\frac{E}{F_{yi}}\right)} \] (A.12.6-7b)

c) For rectangular welded flange or web components supported along one edge
\[ \lambda_r = 0,50 \sqrt{\left(\frac{E}{F_{yi}}\right)} \] (A.12.6-7c)

d) For rectangular rolled or welded web components supported along both edges
\[ \lambda_r = \left[ 1,44 \sqrt{\left(\frac{E}{F_{yi}}\right)} \right] / (0,674 + 0,327 \psi) \quad \text{for } \psi > -1,0 \] (A.12.6-7d)
\[ \lambda_r = \left[ 2,07(1 - \psi) \sqrt{\left(\frac{E}{F_{yi}}\right)} \right] \quad \text{for } \psi \leq -1,0 \] (A.12.6-7e)

Where \( \psi \) is the stress ratio as shown in Table A.12.2-4.

e) For components derived from circular tubes
\[ \lambda_r = 0,102 \frac{E}{F_{yi}} \] [AISC\textsuperscript{[A.12.5-1]}, Table A.12.2-1] (A.12.6-7f)

\( F_{yi} \) = yield stress of component \( i \).

The eccentricity between the elastic and plastic neutral axes, \( e_n \), for class 3 members used in A.12.4 can be calculated as:
A.12.6.2.4 Axial compressive column buckling strength

The representative compression strength of all member classifications subject to flexural buckling should be determined from the following equations:

a) for all grades of steel

\[
P_n = (0.658 \lambda_c^2) P_{pl} \quad \text{for } \lambda_c \leq 1.5 \quad \text{[derived from AISC}^{[A.12.5-1]} \text{, Equation E3-2]} \tag{A.12.6-8a}
\]

\[
P_n = \left( \frac{0.877}{\lambda_c} \right) P_{pl} \quad \text{for } \lambda_c > 1.5 \quad \text{[derived from AISC}^{[A.12.5-1]} \text{, Equation E3-3]} \tag{A.12.6-8b}
\]

b) alternatively, for high strength steels (F_y > 450 MPa), the following may be used:

\[
P_n = (0.7625 \lambda_c^{3.22}) P_{pl} \quad \text{for } \lambda_c \leq 1.2 \tag{A.12.6-9a}
\]

\[
P_n = (0.8608 / \lambda_c^{1.854}) P_{pl} \quad \text{for } \lambda_c > 1.2 \tag{A.12.6-9b}
\]

where

\[
\lambda_c = \frac{KL}{r \pi \left( \frac{P_{pl}}{EA_c} \right)^{0.5}} \quad \text{for max. } KL/r \text{ for } y \text{ and } z \text{ directions} \tag{A.12.6-10}
\]

\[
P_{pl} = \text{compressive axial strength as defined in A.12.6.2.3}
\]

\[
A_c = \text{area of section as defined in A.12.3.5.2 (excluding rack teeth of chords, see A.12.3.1)}
\]

\[
K = \text{effective length factor for } y \text{ or } z \text{ direction, see A.12.4}
\]

\[
L = \text{unbraced length of member for } y \text{ or } z \text{ direction (see A.12.4)}
\]

\[
r = \text{radius of gyration for } y \text{ or } z \text{ direction based on the gross area for sections in classes 1, 2 and 3 and the net area for class 4 (see A.12.3.5.3). The reinforcing effects of rack teeth etc. may be taken into account (see A.12.3.1). When section contains un-reinforced cut-outs, the radius of gyration should be based on the minimum section unless otherwise determined by analysis.}
\]

\[
E = \text{Young's modulus.}
\]

A.12.6.2.5 Bending 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 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 strength

The representative bending strength, $M_b$, is given by the plastic bending moment of the entire section:

$$ M_b = Z_p F_{ymin} \quad (A.12.6-11) $$

where:

- $M_b =$ representative bending moment strength
- $Z_p =$ fully plastic section modulus determined from equation A.12.3-2
- $F_{ymin} =$ minimum yield strength of the cross section 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 strength

The representative bending strength, $M_b$, is obtained by interpolating between the plastic bending moment and the limiting buckling moment:

$$ M_b = M_p - (M_p - M_R) \frac{\lambda - \lambda_p}{\lambda_p - \lambda} \quad (A.12.6-12) $$

where:

- $M_p =$ section plastic moment strength
  = $Z_p F_{ymin}$ as calculated for a class 1 or class 2 cross section
- $Z_p =$ fully plastic section modulus determined from equation A.12.3-2
- $F_{ymin} =$ minimum yield strength of the cross section as defined in A.12.2.2
- $M_R =$ $S_f F_y < M_p \quad (A.12.6-13)$
- $S_f =$ full elastic section modulus for the plane of bending under consideration (A.12.3.3)
- $h =$ subscript referring to the component which produces the smallest value of $M_b$.
- $\lambda =$ $b/t$ or $2R/t$ as applicable for component $h$.
- $\lambda_p$ is determined for component $h$ from: Tables A.12.2-2 to A.12.2-4, and are as below:

  a) For rectangular rolled or welded flange components supported along both edges

  $$ \lambda_p = 1.17 \sqrt{\frac{E}{F_y}} \quad (A.12.6-14a) $$

  b) For rectangular rolled flange or web components supported along one edge

  $$ \lambda_p = 0.37 \sqrt{\frac{E}{F_y}} \quad (A.12.6-14b) $$

  c) For rectangular welded flange or web components supported along one edge

  $$ \lambda_p = 0.33 \sqrt{\frac{E}{F_y}} \quad (A.12.6-14c) $$

  d) For rectangular rolled or welded web components supported along both edges
\[ \lambda_p = \left[ 4.82 \frac{\sqrt{E/F_{yi}}}{(5.12 \alpha - 1)} \right] \] for \( \alpha > 0.5 \) \hspace{1cm} (A.12.6-14d) \\
\[ \lambda_p = \left[ 1.55 \frac{\sqrt{E/F_{yi}}}{\alpha} \right] \] for \( \alpha \leq 0.5 \) \hspace{1cm} (A.12.6-14e)

where \( \alpha \) is a factor that varies depending on the 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.

e) For components derived from circular tubes
\[ \lambda_p = \frac{0.103 E}{F_{yi}} \] \[ \text{[Table A.12.2-1]} \] \hspace{1cm} (A.12.6-14f)

\( \lambda_r \) is determined for component \( h \) from: Tables A.12.2-2 to A.12.2-4, and are as below:

a) For rectangular rolled or welded flange components supported along both edges
\[ \lambda_r = 1.44 \frac{\sqrt{E/F_{yi}}}{\alpha} \] \hspace{1cm} (A.12.6-15a)

b) For rectangular rolled flange or web components supported along one edge
\[ \lambda_r = 0.55 \frac{\sqrt{E/F_{yi}}}{\alpha} \] \hspace{1cm} (A.12.6-15b)

c) For rectangular welded flange or web components supported along one edge
\[ \lambda_r = 0.50 \frac{\sqrt{E/F_{yi}}}{\alpha} \] \hspace{1cm} (A.12.6-15c)

d) For rectangular rolled or welded web components supported along both edges
\[ \lambda_r = \left[ 1.44 \frac{\sqrt{E/F_{yi}}}{\alpha} \right] / (0.674 + 0.327 \psi) \] for \( \psi > -1.0 \) \hspace{1cm} (A.12.6-15d)
\[ \lambda_r = \left[ 2.07(1-\psi)\sqrt{(-\psi)} \right] \sqrt{\frac{E}{F_{yi}}} \] for \( \psi \leq -1.0 \) \hspace{1cm} (A.12.6-15e)

where \( \psi \) is the stress ratio as shown in Table A.12.2-4.

e) For components derived from circular tubes
\[ \lambda_r = \frac{0.22 E}{F_{yi}} \] \[ \text{[Table A.12.2-1]} \] \hspace{1cm} (A.12.6-15f)

\( F_{yi} = \) yield stress of component \( i \).

A.12.6.2.5.4 Class 4 slender section bending strength

The representative bending strength, \( M_b \), of class 4 sections is given by the limiting flexural bending moment:
\[ M_b = S_e F_y \] \hspace{1cm} (A.12.6-16)

where
\[ S_e = \text{reduced elastic section modulus for the plane of bending under consideration} \] \hspace{1cm} (A.12.3.4.3)

A.12.6.2.6 Lateral torsional buckling strength check

The reduced representative moment capacity \( M_e \) due to lateral torsional buckling should be calculated for all members that do not meet the screening checks of either Equation A.12.2-2a or A.12.2-2b for open and closed sections respectively, regardless of the class of section. When this is less than the capacity from A.12.6.2.5 it should be used in the strength checks.

Further guidance on the moment capacity accounting for LTB can be found in the AISC 2005 Specification\[^{[12.5-1]}\], and BS 5400 part 3\[^{[12.5-2]}\]. When the unbraced length \( L_u \) is less than a limiting plastic
length \( L_p \) there is no reduction in the member moment capacity. When the unbraced length \( L_b \) is greater than the limiting unbraced length for inelastic torsional bucking \( L_r \), the member moment capacity is found from the elastic section and a reduced material strength, \( F_{cr} \). When the unbraced length \( L_b \) is between \( L_p \) and \( L_r \) the member moment capacity is found from an interpolation between these capacities.

The formulations for \( L_p \) and \( L_r \) are dependent on the cross-section geometry.

### A.12.6.2.7 Bending strength check

Prismatic members subjected to bending moments, \( M_u \), should satisfy:

\[
M_u \leq M_b / \gamma_{R,Pb} \tag{A.12.6-17}
\]

where

\[
M_u = M_{uy} \text{ or } M_{uz} \text{ the bending moment about member y- and z-axes respectively due to factored actions}
\]

\[
M_b = \text{representative bending moment strength, determined from A.12.6.2.5 and A.12.6.2.6}
\]

\[
\gamma_{R,Pb} = \text{partial resistance factor for bending, 1,1}
\]

### A.12.6.3 Prismatic member combined strength checks

#### A.12.6.3.1 General

There are two different assessment approaches to determine the utilization of structural members, presented in the following clauses:

- Interaction equation approach, which is applicable to all member classifications.
- The plastic interaction surface approach which is applicable to members in class 1 and 2.

#### A.12.6.3.2 Interaction equation approach

Each prismatic structural member should satisfy the following conditions, noting that when the shear is greater than 60 percent of the shear strength, the 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:**

\[
\frac{\gamma_{R,Pa}P_u}{P_p} + \left[ \frac{\gamma_{R,Pa}M_{uy}}{M_{by}} \right]^q + \left[ \frac{\gamma_{R,Pa}M_{uz}}{M_{bz}} \right]^q \leq 1,0 \tag{A.12.6-18}
\]

and beam-column check:

If \( \gamma_{R,Pa}P_u / P_p > 0,2 \)

\[
\frac{\gamma_{R,Pa}P_u}{P_p} + \frac{8}{9} \left[ \frac{\gamma_{R,Pa}M_{uy}}{M_{by}} \right]^q + \left[ \frac{\gamma_{R,Pa}M_{uz}}{M_{bz}} \right]^q \leq 1,0 \quad \text{[after AISC[^1]]}, \text{ Equation H1-1a]}
\]

(A.12.6-19)

else
where

\[
\frac{\gamma_{R,p_b} P_u}{2P_p} \left[ \frac{\gamma_{R,p_b} M_{uay}}{M_{by}} \right]^q + \left[ \frac{\gamma_{R,p_b} M_{uaz}}{M_{bz}} \right]^q \right]^{\frac{1}{q}} \leq 1,0
\]

[after AISC\textsuperscript{[A,12.5-1]}, Equation H1-1b]

(A.12.6-20)

\( P_u \) = applied axial force
\( P_{pl} \) = representative axial strength for the local strength check
\( = P_t \) for members in tension, as defined A.12.6.2.2
\( = P_{pl} \) for members in compression, as defined A.12.6.2.3
\( P_p \) = representative axial strength for the beam column check
\( = P_t \) for members in tension, as defined A.12.6.2.2
\( = P_n \) for members in compression, as defined A.12.6.2.4
\( M_{uay} \) = corrected bending moment about member y-axis due to factored actions from A.12.4
\( M_{uaz} \) = corrected bending moment about member z-axis due to factored actions from A.12.4
\( M_{uay} \) = amplified bending moment about member y-axis due to factored actions from A.12.4
\( M_{uaz} \) = amplified bending moment about member z-axis due to factored actions from A.12.4
\( M_{by} \) = representative moment strength, as defined in A.12.6.2.5 or A.12.6.2.6. When the shear is greater than 60 percent of the shear strength, the moment strength should be reduced parabolically to zero when the shear equals the shear strength \( P_vz \) in Equation A.12.6-29.
\( M_{bz} \) = representative moment strength, as defined in A.12.6.2.5 or A.12.6.2.6. When the shear is greater than 60 percent of the shear strength, the moment strength should be reduced parabolically to zero when the shear equals the shear strength \( P_vy \) in Equation A.12.6-28.
\( \gamma_{R,p_b} \) = partial resistance factor for bending = 1,1
\( \gamma_{R,p_a} \) = partial resistance factor for axial loading
\( = \gamma_{R,p_t} \) for axial tension = 1,05
\( = \gamma_{R,p_c} \) for axial compressive strength = 1,1
\( \eta \) = Exponent for biaxial bending, a constant dependent on the member cross section geometry, determined as follows:

i) For purely circular tubular members \( \eta = 2,0 \)

ii) For solid or hollow rectangular sections \( \eta = \frac{5}{3} \)

iii) For doubly symmetric open section members \( \eta = 1,0 \)

iv) For all geometries, a conservative value of \( \eta = 1,0 \) may be used.
Annex F presents an approach to determining the value of \( \eta \) by manual calculation.

### A.12.6.3.3 The 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\[^{A.12.5-5}\] and used to determine the member utilization. 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. The assessor should note 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.

A measure of the interaction ratio can then be obtained as the ratio between the vector lengths from the functional origin to the member forces and 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 example conservative interaction equations and curves for generic families of chord cross sections based on ultimate strengths \( P_y \), \( M_{px} \), and \( M_{py} \). It is necessary to introduce the resistance factors. This is achieved by defining:

\[
P_y = F_1 P_n \gamma_{R,Pa} \tag{A.12.6-21}
\]
\[
M_{py} = F_2 M_{uay} \gamma_{R,Pb} \tag{A.12.6-22}
\]
\[
M_{pz} = F_2 M_{uaz} \gamma_{R,Pb} \tag{A.12.6-23}
\]

where

\( M_{uay}, M_{uaz} \) = amplified bending moment about member y- and z-axis due to factored actions from A.12.4 calculated with \( B \geq 1,0 \).

\( F_1 = 1,0 \) unless alternative values are justified by analysis.

\( F_2 = 1,0 \) unless alternative values are justified by analysis and when the shear force is less than 60 percent of the shear strength \( (P_{vy}, P_{vz} \text{ in A.12.6.3.4}) \).

\( \gamma_{R,Pa} \) = partial resistance factor for axial loading

\( \gamma_{R,Pb} \) = partial resistance factor for bending = 1,1

\( \gamma_{R,Pt} \) = partial resistance factor for axial tension = 1,05

\( \gamma_{R,Pc} \) = partial resistance factor for axial compressive strength = 1,1

### A.12.6.3.4 Beam shear

Prismatic members subjected to beam shear forces should satisfy the following:

\[
V_y \leq P_{vy} / \gamma_{R,Pv} \tag{A.12.6-24}
\]
\[
V_z \leq P_{vz} / \gamma_{R,Pv} \tag{A.12.6-25}
\]

where

\( V_y, V_z \) = beam shear due to factored actions in the local y and z directions

\( P_{vy}, P_{vz} \) = representative shear strength in the local y and z directions
\[ A_v = \frac{A_{\text{min}}}{\sqrt{3}} \]

- \( A_v \) = effective shear area in the direction being considered
- \( \gamma_{R,P_v} \) = partial resistance factor for beam shear strength = 1,1

### Table A.12.6-1 Effective shear area for various cross sections

<table>
<thead>
<tr>
<th>Section</th>
<th>Effective shear area, ( A_v )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Rolled I, H and channel sections, load parallel to web</td>
<td>( tD )</td>
</tr>
<tr>
<td>Welded I sections, load parallel to web</td>
<td>( td )</td>
</tr>
<tr>
<td>Rectangular hollow sections, load parallel to webs</td>
<td>( \frac{AD}{D+D} )</td>
</tr>
<tr>
<td>Welded box sections, load parallel to web</td>
<td>( 2td )</td>
</tr>
<tr>
<td>Rolled Tee-sections, load parallel to web</td>
<td>( td )</td>
</tr>
<tr>
<td>Welded Tee-sections, load parallel to web</td>
<td>( t(D-T) )</td>
</tr>
<tr>
<td>Circular hollow sections</td>
<td>0.5( A )</td>
</tr>
<tr>
<td>Solid bars and plates</td>
<td>0.9( A )</td>
</tr>
<tr>
<td>Closed sections with inclined plates</td>
<td>( 0.9 \sum (\cos(\theta) A_{oi}) )</td>
</tr>
</tbody>
</table>

Where

\[ T = \text{flange thickness of a welded T-section} \]
\[ t = \text{web thickness} \]
\[ D = \text{overall depth} \]
\[ d = \text{web depth. For rolled sections, measured with respect to root radii. For welded sections, measured between inside faces of flanges.} \]
\[ B = \text{overall breadth} \]
\[ A = \text{area of cross-section} \]
\[ A_{oi} = \text{area of rectilinear component} \]
\[ \theta = \text{angle between the shear force direction being considered and the larger dimension of the cross-section of component} \]

### A.12.6.3.5 Torsional shear

Closed section prismatic members subjected to torsional shear forces should satisfy the following:

\[ T \leq \frac{T_v}{\gamma_{R,P_v}} \]

where

\[ T = \text{torsional moment due to factored actions} \]
\[ T_v = \text{representative torsional strength} = I_p \frac{F_{\text{min}}}{r \sqrt{3}} \]
\[ I_p = \text{polar moment of inertia} \]
Open section 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 significantly 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, tubular joints can be assessed in accordance with ISO 19902 subclause 24.9.2.2.2 (Connections) and non-tubular joints by rational analysis. The internal forces (action effects) due to factored actions should be determined in accordance with subclause 8.8, rather than using the factors in 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 applied actions. The joint strength need not necessarily meet or exceed the full member strength. Guidance on non-tubular joint strength can be found in other clauses of ISO 19902 and 19901-3.

A.13 Acceptance checks

No guidance offered.
## Annex B
(normative)

### Summary of partial action and resistance factors

<< Note from WG7 to the reader: This draft international standard has been prepared for site-specific analysis of the jack-up platform using best available practices from current technology. The methods presented are being subjected to a comprehensive benchmarking process to confirm that results are reasonably consistent with current best practice. Until this process is complete, the standard should not be utilized as an exclusive resource for jack-up site assessment. >>

<table>
<thead>
<tr>
<th>Symbol</th>
<th>Description</th>
<th>Factor</th>
<th>Ref. Clause</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\gamma_f$</td>
<td>Factor to be applied to the Fixed Actions $G$</td>
<td>1,0</td>
<td>8.8.1</td>
</tr>
<tr>
<td>$\gamma_{f,V}$</td>
<td>Factor to be applied to the actions due to variable load $G_v$</td>
<td>1,0</td>
<td>8.8.1</td>
</tr>
<tr>
<td>$\gamma_{f,E}$</td>
<td>Factor to be applied to the deterministic ULS storm action $E_o$ (used with 50 year independent extreme values)</td>
<td>1,15</td>
<td>8.8.1.2</td>
</tr>
<tr>
<td>$\gamma_{f,V}$</td>
<td>Factor to be applied to the deterministic ULS storm action $E_o$ (used with 100 year joint probability metocean data)</td>
<td>1,25</td>
<td>8.8.1.2</td>
</tr>
<tr>
<td>$\gamma_{f,D}$</td>
<td>Factor to be applied to the stochastic ULS storm actions $E_o$ using factored metocean parameters determined in accordance with A.10.5.3.2 (see Note 2).</td>
<td>1,0</td>
<td>8.8.1.3</td>
</tr>
<tr>
<td>$\gamma_{f,V}$</td>
<td>Factor to be applied to the inertial action induced by the ELE ground motions in earthquake analysis</td>
<td>0,9</td>
<td>8.8.1.4</td>
</tr>
<tr>
<td>$\gamma_{f,V}$</td>
<td>Factor to be applied to the inertial action induced by the ALE ground motions in earthquake analysis</td>
<td>1,0</td>
<td>8.8.1.4</td>
</tr>
<tr>
<td>$\gamma_{R,S}$</td>
<td>Factor on the Dynamic Action $D_o$ in combination with $\gamma_{f,E}$</td>
<td>1,0</td>
<td>8.8.1</td>
</tr>
<tr>
<td>$\gamma_{R,H}$</td>
<td>Resistance Factor for spudcan strength</td>
<td>1,15</td>
<td>13.4</td>
</tr>
<tr>
<td>$\gamma_{R,H}$</td>
<td>Resistance Factor for vertical holding system between hull and leg</td>
<td>1,15</td>
<td>13.5</td>
</tr>
<tr>
<td>$\gamma_{R,OTM}$</td>
<td>Resistance Factor for overturning</td>
<td>1,05</td>
<td>13.8</td>
</tr>
<tr>
<td>$\gamma_{R,PRE}$</td>
<td>Resistance Factor for preload</td>
<td>1,1</td>
<td>A.9.3.6.2</td>
</tr>
<tr>
<td>$\gamma_{R,Hc}$</td>
<td>Partial resistance factor for horizontal foundation capacity for effective stress - sand/drained</td>
<td>1,25</td>
<td>A.9.3.6.2</td>
</tr>
<tr>
<td>$\gamma_{R,Hc}$</td>
<td>Partial resistance factor for horizontal foundation capacity total stress - clay/undrained</td>
<td>1,56</td>
<td>A.9.3.6.2</td>
</tr>
<tr>
<td>$\gamma_{R,VH}$</td>
<td>Partial resistance factor for foundation capacity, maximum bearing area not mobilized.</td>
<td>1,1</td>
<td>A.9.3.6.4</td>
</tr>
<tr>
<td>Symbol</td>
<td>Description</td>
<td>Factor</td>
<td>Ref. Clause</td>
</tr>
<tr>
<td>---------</td>
<td>------------------------------------------------------------------</td>
<td>--------</td>
<td>-------------</td>
</tr>
<tr>
<td>$\gamma_{R,Ti}$</td>
<td>Resistance Factor for tubular axial tensile strength</td>
<td>1,05</td>
<td>A.12.5</td>
</tr>
<tr>
<td>$\gamma_{R,Tc}$</td>
<td>Resistance Factor for tubular axial compressive strength</td>
<td>1,15</td>
<td>A.12.5</td>
</tr>
<tr>
<td>$\gamma_{R,Tb}$</td>
<td>Resistance Factor for tubular bending strength</td>
<td>1,05</td>
<td>A.12.5</td>
</tr>
<tr>
<td>$\gamma_{R,Tv}$</td>
<td>Resistance Factor for tubular shear strength</td>
<td>1,05</td>
<td>A.12.5</td>
</tr>
<tr>
<td>$\gamma_{R,Pi}$</td>
<td>Resistance Factor for prismatic axial tensile strength</td>
<td>1,05</td>
<td>A.12.6</td>
</tr>
<tr>
<td>$\gamma_{R,Pcl}$</td>
<td>Resistance Factor for prismatic local axial compressive strength</td>
<td>1,1</td>
<td>A.12.6</td>
</tr>
<tr>
<td>$\gamma_{R,Pc}$</td>
<td>Resistance Factor for prismatic global axial compressive strength</td>
<td>1,1</td>
<td>A.12.6</td>
</tr>
<tr>
<td>$\gamma_{R,Pb}$</td>
<td>Resistance Factor for prismatic bending strength</td>
<td>1,1</td>
<td>A.12.6</td>
</tr>
<tr>
<td>$\gamma_{R,Pv}$</td>
<td>Resistance Factor for prismatic shear strength</td>
<td>1,1</td>
<td>A.12.6</td>
</tr>
</tbody>
</table>

NOTE 1 Values given in this table are normative. The reference clauses provide the methods of application and the factors are specifically tied to the calculation methodologies given in each reference clause.

NOTE 2 The metocean partial factors used in the quasi-static stochastic analysis are determined through an iterative process. The process 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 $\sqrt{1,15}$ or $\sqrt{1,25}$ as applicable.
Annex C
(informative)

Structural modelling and response analysis - additional information
C.1 Guidance on 8.5: Modelling the leg/hull connections

![Diagram of leg/hull connection component combinations]

**F&G**
- L780 II
- JU 2000
- Alpha 350
- Super M2
- Universal M class

**Hitachi**
- Giant class
- GustoMSC
- CJ 46
- CJ 50

**Keppler FELS**
- KFELS Mod V
- KFELS Mod VI
- A Class
- B Class
- N Class

**LeTourneau**
- Super GORILLA
- Super GORILLA XL
- Jaguar 250-C

**GustoMSC**
- CJ 36
- CJ 46
- CJ 50
- CJ 62
- CJ 70

**Modec**
- 300C
- 400

**Baker Marine**
- Pacific 375
- 200 I.C.

**CFEM**
- 2005
- 2600

**LeTourneau**
- 53
- 64
- 82-SD-C
- 116C
- Super 116
- Super 116E
- Super 300
- Gorilla

**Levingston**
- 111C

**GustoMSC**
- Gusto designs

**Example of Units in this category**

**Figure C.1-1- Sample Leg/hull connection component combinations**
C.2 Guidance on A.10.4.2.1: General

If such a capability (a finite element structural model containing the mass and stiffness properties of the jack-up) is not available, due to the fact that the mass of the hull dominates the mass distribution, the global dynamic behaviour of the jack-up may in some cases be determined from an idealized single degree-of-freedom system and thus the fundamental mode period may be estimated from the system described by:

— an equivalent mass representing the mass of the jack-up and its distribution as referred to in 10.4.2.3 the equivalent mass is equal to the mass of the hull plus a contribution from the mass of the legs, including added mass, and is located at the centre of gravity of the hull.

— an equivalent spring representing the combined effect of the overall (global) structural stiffness including stiffness contributions from bending, shear deformation and axial straining of the legs, the leg to hull connections, the hull and the spudcan-foundation interface (if applicable).

The period is determined from the following equation applied to one leg:

\[
T_n = \frac{2\pi}{\sqrt{\left(\frac{M_e}{K_e}\right)}}
\]  

(C.2-1)

where

\[
T_n = \text{highest (or first mode) natural period.}
\]

\[
M_e = \text{effective mass associated with one leg.}
\]

\[
= \frac{M_{hull}}{N} + M_{la} + \frac{M_{lb}}{2}
\]  

(C.2-2)

\[
M_{hull} = \text{full mass of hull including maximum variable load.}
\]

\[
N = \text{number of legs.}
\]

\[
M_{la} = \text{mass of leg above lower guide (in the absence of a clamping mechanism) or above the centre of the clamping mechanism.}
\]

\[
M_{lb} = \text{mass of leg below the point described for } M_{la}, \text{ including added mass for the submerged part of the leg ignoring spudcan. The added mass may be determined as } A_e \rho (C_{Me} - 1) \text{ per unit length of one leg (for definitions of } A_e \text{ and } C_{Me} \text{ see A.7.3.2.3)); } \rho = \text{mass density of water.}
\]

\[
K_e = \text{effective stiffness associated with one leg (for derivation, refer to ISO TR 19905-2 A.10.4.2.2).}
\]

\[
= \frac{3EI}{L^3} \left[ 1 - \frac{P}{P_c} \right]
\]  

(C.2-3)

When the soil rotational stiffness \( K_{rs} \) at the spudcan-foundation interface is zero this may be re-written:
\[ K_e = \frac{3EI}{L^3} \left[ 1 - \frac{P}{P_k} \right] \]  
\[ 1 + \frac{12F_gI_v}{AF_vY^2} + \frac{3EI}{F_iLK_{th}} + \frac{7.8I}{A_sF_iL^2} \]  
(C.2-4)

\( K_{rs} \) = rotational spring stiffness at spudcan-foundation interface.

\( K_{th} \) = rotational stiffness representing leg to hull connection stiffness (see below).

\( F_r \) = factor to account for hull bending stiffness.

\[ = \frac{1}{1 + \frac{YK_{th}}{2EI_H}} \]  
(C.2-5)

\( I_H \) = representative second moment of area of the hull girder joining two legs about a horizontal axis normal to the line of environmental action.

\( E \) = Young’s modulus for steel.

\( A \) = axial area of one leg (equals sum of effective chord areas, including a contribution from rack teeth - see Note to A.8.3.3).

\( A_s \) = effective shear area of one leg (see Figure A.8.3-1).

\( I \) = second moment of area of the leg (see Figure A.8.3-1), including a contribution from rack teeth (see Note to A.8.3.3).

\( Y \) = distance between centre of one leg and line joining centres of the other two legs (3 leg jack-up).

\[ = \text{distance between windward and leeward leg rows for direction under consideration (4 leg jack-up)} \]

\( F_g \) = geometric factor.

\( = 1,125 \) (3 leg jack-up), 1.0 (4 leg jack-up)

\( F_v \) = factor to account for vertical soil stiffness, \( K_{vs} \), and vertical leg-hull connection stiffness, \( K_{vh} \) (see below).

\[ = \frac{1}{1 + \frac{EA}{LK_{vs}} + \frac{EA}{LK_{vh}}} \]  
(C.2-6)

\( F_h \) = factor to account for horizontal soil stiffness, \( K_{hs} \), and horizontal leg-hull connection stiffness, \( K_{hh} \) (see below).

\[ = \frac{1}{1 + \frac{EA_s}{2.6LK_{hs}} + \frac{EA_s}{2.6LK_{hh}}} \]  
(C.2-7)

\( L \) = length of leg from the seabed reaction point (see A.8.6.2) to the point separating \( M_{ia} \) and \( M_b \) (see above).
\[ P = \text{the mean force due to vertical fixed load and variable load acting on one leg.} \]

\[ = \frac{M_{\text{null}} g}{N} \quad \text{(C.2-8)} \]

\[ g = \text{acceleration due to gravity.} \]

\[ P_E = \text{Euler buckling force of one leg.} \]

\[ = \alpha 2EI \quad \text{(C.2-9)} \]

\[ \alpha = \text{the minimum positive non-zero value of } \alpha L \text{ satisfying:} \]

\[ \tan(\alpha L) = \frac{(K_{rs} + K_{rh})\alpha EI}{(dEI)^2 - (K_{rs} K_{rh})} \quad \text{(C.2-10)} \]

Thus:

when \( K_{rs} = 0 \) and \( K_{rh} = \infty \), \( \alpha L = \pi/2 \) and hence:

\[ P_E = \frac{\pi^2 EI}{4L^2} \quad \text{(C.2-11a)} \]

when \( K_{rs} = \infty \) and \( K_{rh} = \infty \), \( \alpha L = \pi \) and hence

\[ P_E = \frac{\pi^2 EI}{L^2} \quad \text{(C.2-11b)} \]

The hull to leg connection springs, \( K_{vh} \) and \( K_{hh} \) represent the interaction of the leg with the guides and supporting system and account for local member flexibility and frame action. They should be computed with respect to the point separating \( M_{la} \) and \( M_{lb} \), as described above. The following approximations may be applied:

\[ K_{vh} = \infty \]

\[ K_{hh} = \text{effective stiffness due to the series combination of all vertical pinion or fixation system stiffnesses, allowing for combined action with shock-pads, where fitted.} \]

Jack-up with fixation system:

\[ K_{rh} = \text{combined rotational stiffness of fixation systems on one leg.} \]

\[ = F_n h 2k_i \quad \text{(C.2-12a)} \]

where

\[ F_n = 0.5, \text{ three chord leg;} = 1.0, \text{ four chord leg} \]

\[ h = \text{distance between chord centres.} \]

\[ k_i = \text{combined vertical stiffness of all fixation system components on one chord.} \]

Jack-up without fixation system:
\[ K_{rh} = \text{rotational stiffness allowing for pinion stiffness, leg shear deformation and guide flexibility.} \]

\[ = F_n h 2k_j + \frac{k_u d^2}{1 + (2.6 k_u d/E A_s)} \quad \text{(C.2-12b)} \]

where

\[ h = \text{distance between chord centres (opposed pinion chords) or pinion pitch points (single rack chords).} \]

\[ k_j = \text{combined vertical stiffness of all jacking system components on one chord.} \]

\[ d = \text{distance between upper and lower guides.} \]

\[ k_u = \text{total lateral stiffness of upper guides with respect to lower guides.} \]

\[ A_s = \text{effective shear area of leg.} \]

The above equations for estimating the fundamental natural period are approximate and ignore the following effects:

- more realistic representation of possible fixity at the spudcan-foundation interface in the form of (coupled) horizontal, vertical and rotational spring stiffnesses.
- three dimensional influences of the system as compared with the two-dimensional single leg model.

C.3 Guidance on A.10.4.2.4: Variability in natural period

In selecting the wave excitation period, consideration should be given to the position of the natural period(s) in relation to the cancellation and reinforcement points in the global wave loading of the jack-up which is important for the magnitude of any dynamic wave magnification. Cancellations and reinforcements in global loading are due to spatial separation of the wave load attracting legs and may be different for different wave directions. The global loading may be characterized by the total horizontal wave loading or overturning moment; cancellation and reinforcement of points for these may appear at slightly different wave periods.

In order to avoid the possibility of under-conservative dynamic amplification factors, it is important to investigate the relationship between the jack-up natural period and the cancellation and reinforcement points in the transfer functions relating wave height to base shear and overturning moment. The range of possible natural period(s) should be bracketed and compared with the relevant cancellation/reinforcement points in the global wave loading. 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, avoiding maximum dynamic amplification to coincide with minimum metocean loading. Figure A.10.4-1 may be used for a first evaluation of the position of the calculated natural period(s), but it is recommended that the definitive selection of natural period(s) be based on the shape of the global horizontal wave loading (base shear) and overturning moment transfer functions calculated for the actual application under consideration.

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.

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 spudcan moment fixity, then the adjustment would most logically be made by varying the degree of rotational fixity at the seabed.
To minimise cancellation effects, it is suggested that the dynamic analysis may be carried out for a single wave heading along an axis which is neither parallel nor normal to a leg line. Thus, for a 3-legged jack-up with equilateral leg positions and a single bow leg, suitable analysis headings would be with the environment approaching from approximately 15° or 45° off the bow. The dynamic amplification factors (DAF's) should be determined for one, or both, of these headings, with suitably adjusted natural period. The DAF’s (or more conservative DAF’s) may then be applied to the final quasi-static analysis for all headings and hull weight cases with, when applicable, foundation stiffness reduction (see A.9.3.4.2).

C.4 Guidance on A.10.4.3.3: SDOF hysteretic damping

NOTE <<This section is potentially subject to some change based on further benchmarking of the method.>>

The following hysteretic damping may be used only in conjunction with an SDOF analysis. Where a linear foundation model with stiffness reduction according to A.9.3.4.2.1 is adopted, the hysteretic damping may be added. Hysteretic damping should be only in combination with nonlinear rotational stiffness reduction and should not be used in combination with initial stiffness.

Foundation hysteretic damping is a consequence of the hysteretic behaviour of the foundation soils. For clay soils, whenever significant foundation nonlinearity is present, an additional damping component may be added to system damping to account for this phenomenon. This foundation hysteretic damping component can be included implicitly in a detailed nonlinear dynamic analysis embodying hysteretic spudcan foundation elements, or it may be calculated explicitly in the case of a simpler quasi-static analysis, as follows:

1) The entire structure should be modelled (e.g., via a bar stool model). The model should be linear except for the inclusion of foundation nonlinearities (e.g., via the use of hysteretic spudcan foundation elements or progressive stiffness reduction) and P-Δ effects,

2) This model should be used to produce a force vs. deflection (backbone) curve for the horizontal force, $F$ (equal to the amplitude of the extreme load cycle, including the effects of dynamic amplification), vs. the horizontal deflection, $x$, both at the effective centre of combined storm and inertial loading,

3) The hysteretic damping, $D$, as a function of deflection should be developed from the $F$ vs. $x$ (backbone) curve according to the definition,

$$D = \frac{2}{\pi} \frac{\int_0^x F \, dx}{\left[ \frac{2}{F^2} \left(-1 + \int_0^x F \, dx \right) \right]}$$

(C.4-1)

due to the magnitude of $F$ equal to the amplitude of the extreme load cycle, including the effects of dynamic amplification, which implies multiple or iterative analysis to determine consistent values of $F$ and $D$.

NOTE << It has been suggested that a Figure be added, and that more background to the method be provided in ISO/TR 19905-2, which could include a more detailed procedure, or an example from the calibration. >>

4) The hysteretic damping $D$ (a fraction of critical damping) should be added to the small-strain soil material damping used in determining the SDOF DAF, $K_{\text{DAF,SDOF}}$.

5) The hysteretic damping should be determined for each loading direction to be considered in the assessment.
C.5 Guidance on A.10.5.3.2: Methods for determining the MPME

C.5.1 Guidance on A.10.5.3.2.1: Fit Weibull distribution to results of a number of time-domain simulations to determine responses at required probability level and average the results.

This procedure requires a suitable length time domain simulation record for each quantity of interest. The input sea state record should be checked for ‘Gaussianity’. Guidance is given in Tables A.7.3-4 and A.10.5-2. The procedure comprises the following steps.

Step 1

The signal record is first analyzed to calculate the mean, $\mu_R$, as:

$$\mu_R = \frac{\sum_{i=1}^{n} R(t_i)}{n} \quad (C.5.1-1)$$

where

- $R(t_i)$ = time history of signal
- $t_i$ = time points
- $n$ = number of useable time points in simulation (discounting the run-in)

Step 2

The individual point-in-time maxima are next extracted according to the following criteria:

A maximum occurs at $t_i$ if:

$$R(t_{i-1}) < R(t_i) \text{ and } R(t_{i+1}) \leq R(t_i) \quad (C.5.1-2)$$

Suppose $N_{\text{max}}$ maxima are found in the extraction.

Step 3

From the $N_{\text{max}}$ maxima, the mean of the signal, $\mu_R$, is subtracted and the maxima $R_{\text{max}(i)}$ are ranked into 20 blocks having mid-points in ascending order. The blocks all have the same width and the upper bound of block 20 is taken as being $1.01 \times$ the largest value, the lower bound of the first block being zero. A distribution of maxima observations 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 $R$. If each block has $n_i$ maxima, the cumulative probability $F_i$ to be plotted against the mid point for block $i$ is then given by:

$$F_i = \left(1 + \sum_{j=0}^{i} n_j \right) \frac{\sum_{j=0}^{i} n_j}{(N_{\text{max}} + 1)} \quad (C.5.1-3)$$

where $n_0 = 0$.

Step 4.a
A Weibull distribution is fitted against the cumulative distribution of the maxima as defined under Step 3 (see Steps 4.b to 4.d). The 3-parameter Weibull cumulative distribution function is defined as:

\[
F(R; \alpha, \beta, \gamma) = 1 - \exp \left[ -\left( \frac{R - \gamma}{\alpha} \right)^\beta \right]
\]  
(C.5.1-4)

where

\[
F(R; \alpha, \beta, \gamma) = \text{probability of non-exceedence}
\]

\[
\alpha = \text{scale parameter}
\]

\[
\beta = \text{slope parameter}
\]

\[
\gamma = \text{threshold parameter}
\]

and \( \alpha, \beta, (R - \gamma) > 0 \)

Step 4.b

Only data points \( R_{(\text{max},i)} \), corresponding to a probability of non-exceedence greater than a threshold value of 0.2 are used to fit the Weibull distribution, i.e. only the points:

\[
\left[ R_{(\text{max},i)} \right] \left[ \frac{N_{\text{max}} - i + 1}{N_{\text{max}}} \right] \text{ for } i > 0.2 \times N_{\text{max}}
\]  
(C.5.1-5)

Notice that \( R_{(\text{max},i)} \) are in ascending order.

Step 4.c

For each of these points, the deviations between the Weibull distribution and the values \( R_{(\text{max},i)} \) (transformed to Weibull scales) are calculated as:

\[
\delta_i = \ln[ -\ln \left( 1 - F(R_{(\text{max},i)}, \alpha, \beta, \gamma) \right) \} - \beta \left[ \ln(r_{(\text{max},i)} - \gamma) - \ln(\alpha) \right]
\]  
(C.5.1-6)

Step 4.d

The parameters \( \alpha, \beta, \gamma \) are now estimated by a nonlinear least square technique, i.e.

\[
\sum_{i=0.2N_{\text{max}}}^{N_{\text{max}}} \delta_i^2 \text{ is minimized}
\]  
(C.5.1-7)

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 \( R_{\text{MPM}} \) is found as the value of \( R \) for which:

\[
F(R; \alpha, \beta, \gamma) = 1 - \frac{1}{N_{\text{max}} \times 3 \text{ hours} \over \text{simulation duration}}
\]  
(C.5.1-8)
Step 6

The total extreme MPM value, $R_{MPME}$ is found as:

$$R_{MPME} = \mu_R + R_{MPM}$$  \hspace{1cm} (C.5.1-9)

where $\mu_R$ = the mean value of $R$ established in Step 1 $R_{MPM}$ = the MPM value (excluding the mean) established in Step 5.

Step 7

The procedure is repeated for each required response parameter.

The basic assumption of this method is that the 3-hour extreme values follow a Gumbel distribution:

C.5.2 Guidance on A.10.5.3.2.2: Fit Gumbel distribution to histogram of peak responses from a number of time-domain simulations to determine responses at required probability level.

The basic assumption of this method is that the 3-hour extreme values follow a Gumbel distribution:

$$F_{3h}^{-1}(x) = \exp\left[-\exp\left(-\frac{x-\psi}{\kappa}\right)\right]$$  \hspace{1cm} (C.5.2-1)

where

- $F_{3h}(x)$ = the probability that the 3-hour maximum does not exceed value $x$.
- $\psi$ = location parameter
- $\kappa$ = scale parameter

The following steps are followed for each required response parameter:

Step 1

Extract maximum (and minimum) value for each of 10 3-hour response signal records.

Step 2

A Gumbel distribution is fitted through these 10 maxima/minima. This is done using the maximum likelihood method, yielding $\psi$ and $\kappa$.

Step 3

The $MPME$ is found according to:

$$MPME = \psi - \kappa \ln \left\{ -\ln\left( F_{3h}^{MPME} \right) \right\}$$  \hspace{1cm} (C.5.2-2)

with;

$$F_{3h}^{MPME} = 0.37$$
The 0.37 lower quantile is used because the extreme of recurrence of once in 3 hours has a probability of exceedence of 0.63 (= 1 - 0.37). In this case it can be seen that:

\[ MPME = \psi \]

**Step 4**

The procedure of Step 3 is similarly applied for minima.

**C.5.3 Guidance on A.10.5.3.2.3:** Apply 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 = 1.86 x significant wave height). For general nonlinear, non-Gaussian, finite band-width processes, approximate methods are used to generate the probability density function of the process. The method proposed by Winterstein, Reference [C.5-1], fits a Hermite polynomial of Gaussian processes to transform the nonlinear, non-Gaussian process into a mathematically tractable probability density function. This has been further refined by Jensen, Reference [C.5-2], for processes with large kurtosis.

This procedure requires a suitable length time domain simulation record for each quantity of interest. The input sea state record should be checked for ‘Gaussianity’. Guidance is given in Tables A.7.3-4 and A.10.5-2. The calculation procedure to determine the maximum of a time series, \( R(t) \), in duration \( T \) is as follows:

**Step 1**

Calculate the following quantities of the time series for the parameter under consideration:

\[ \mu = \text{mean of the process} \]
\[ \sigma = \text{standard deviation} \]
\[ \alpha_3 = \text{skewness} \]
\[ \alpha_4 = \text{kurtosis} \]

**Step 2**

Hence construct a standardised response process, \( z = (R - \mu)/\sigma \). Using this standardised process, calculate the number of zero-upcrossings, \( N \). In lieu of an actual cycle count from the simulated time series, \( N = 1000 \) may be assumed for a 3-hour simulation.

**Step 3**

Compute the following quantities from the characteristics of the response parameters identified earlier:

\[ h_3 = \alpha_3 \div \left[ 4 + 2\sqrt{(1 + 1.5(\alpha_4 - 3))} \right] \] (C.5.3-1)
\[ h_4 = \sqrt{(1 + 1.5(\alpha_4 - 3))} \div 18 \] (C.5.3-2)
\[ K = \left[ 1 + 2h_3^2 + 6h_4^2 \right]^{1/2} \] (C.5.3-3)

It is necessary to seek a more accurate result by determining the solution of the following equations for \( C_1 \), \( C_2 \) and \( C_3 \):

\[ \sigma^2 = C_1^2 + 6C_1C_3 + 2C_2^2 + 15C_3^2 \] (C.5.3-4)
\begin{align*}
\alpha_3^3 &= C_2 (6C_1^2 + 8C_2^2 + 72C_1C_3 + 270C_3^2) \quad (C.5.3-5) \\
\alpha_4^4 &= 60C_2^4 + 3C_1^4 + 10395C_3^4 + 60C_1^2C_2^2 + 4500C_2^2C_3^2 + 630C_1^2C_3^2 + 936C_1C_2^2C_3 + 3780C_1C_3^3 + 60C_1^3C_3 \quad (C.5.3-6)
\end{align*}

using as initial guesses:
\begin{align*}
C_1 &= \sigma K (1 - 3h_4) \quad (C.5.3-7) \\
C_2 &= \sigma Kh_3 \quad (C.5.3-8) \\
C_3 &= \sigma Kh_4 \quad (C.5.3-9)
\end{align*}

with \( \sigma, K, h_3 \) and \( h_4 \) from above. Following the solution for \( C_1, C_2 \) and \( C_3 \), the values for \( K, h_3 \) and \( h_4 \) are computed as follows:
\begin{align*}
K &= \frac{C_1 + 3C_3}{\sigma} \quad (C.5.3-10) \\
h_3 &= \frac{C_2}{\sigma K} \quad (C.5.3-11) \\
h_4 &= \frac{C_3}{\sigma K} \quad (C.5.3-12)
\end{align*}

**Step 4**

The most probable value, \( U \), of the transformed process is computed by the following equation:
\begin{align*}
U &= \sqrt{2\log_8\left(\frac{N \cdot 3 \text{ hours}}{\text{simulation time (in hours)}}\right)} \quad (C.5.3-13)
\end{align*}

Where \( U \) is a Gaussian process of zero mean, unit variance.

**Step 5**

The most probable maximum, transformed back to the standardised variable, \( z \), is then given by:
\begin{align*}
z_{\text{MPM}} &= K \left[ U + h_3(U^2 - 1) + h_4(U^3 - 3U) \right] \quad (C.5.3-14)
\end{align*}

**Step 6**

Finally, the MPME in the period \( T \), for the response under consideration, can be computed from the following equation:
\begin{align*}
R_{\text{MPME}} &= \mu + \sigma z_{\text{MPM}} \quad (C.5.3-15)
\end{align*}
Annex D
(informative)

Foundations: Recommendations for the acquisition of site specific geotechnical data
D.1 Recommendations for the acquisition of site specific geotechnical data

NOTE << The text for this clause has only just been received and has not been fully edited for compliance with ISO directives. >>

This Annex, based on a report by the InSafeJIP, Reference [D.1-1], provides recommendations for the acquisition of site specific geotechnical data for jack-up foundation assessment purposes.

It is assumed that regional geological information is available, a geological desk study has been conducted and that a site geophysical survey has been performed in advance of the offshore geotechnical works as this information is required in order to plan and optimise the geotechnical site investigation work-scope.

Regional geohazards, if present, are unlikely to be identified using the site specific geotechnical data in isolation, which should be integrated with the local geophysical survey, regional geological data and any other information which would be useful in assessing the potential presence of regional geohazards.

The primary objective of the geotechnical works is to acquire adequate data in order to minimise the seabed risk, and allow for risk avoidance or mitigation should it be necessary.

The geotechnical risks associated with jack-up operations are listed elsewhere, Reference [D.1-2] and Table A.6.5-1.

Ideally during a field development planning stage adequate consideration should be given to the acquisition and integration of geophysical and geotechnical data prior to the installation of any facilities. If jack-ups are to be used for work-over or as installation support facilities throughout the field life then due consideration has to be given to the positioning of these units and the implication of the seabed depressions and zones of disturbed soil (footprints) caused during spudcan installation upon other operations. The range of jack-up designs suitable for operating within the field should be considered and the implications of each on each others operation, in terms of spudcan-footprint interaction, should be assessed.

Well conductor slot access from jack-ups positioned at different platform faces may be possible, and the positioning of seabed architecture to allow jack-up positioning flexibility is often advantageous.

Site specific geophysical survey and geotechnical data acquired should be applicable for both fixed structure foundation design and jack-up foundation assessment.

Although there may be exceptions it is generally necessary to acquire geotechnical data to a depth below mudline of either 30m, or the anticipated spudcan penetration depth plus 1,5 spudcan diameters, whichever is the greatest.

It is also recognised that it may not be possible to conduct an optimally designed offshore work-scope so that a compromise solution may be necessary. Such factors may include:

- Availability of dedicated geotechnical site investigation vessel with experienced personnel
- Availability of specific geotechnical systems and tools
- Weather conditions precluding or reducing offshore operations
- Site accessibility

The following table lists various site conditions and provides recommendations for offshore geotechnical site investigation works which can be required in order to conduct a jack-up foundation site assessment for both “open” and “work-over” locations are considered. These tables address the offshore PCPT and borehole requirements only. PCPT data acquisition and processing requirements, and soil laboratory testing requirements and specifications are discussed elsewhere, Reference [D.1-3], and A.6.5.
These recommendations should only be used for guidance and do not imply any legislative requirements, responsibilities or guarantee of applicability.

<table>
<thead>
<tr>
<th>Site Conditions &amp; Considerations</th>
<th>Geotechnical Site Investigation Recommendations</th>
</tr>
</thead>
</table>
| **A** Regional and local geology well understood  
Near-surface conditions suitable for jack-up operations  
High quality geophysical survey data available; sub-bottom profiling data tied back to a geotechnical borehole(s) and / or local jack-up installation location(s)  
Mature jack-up operating province where foundation issues are not expected  
Desk study corroborates geophysical data  
Adverse foundation performance risk extremely remote and any potential risk is expected to manageable | Acquisition of site specific geotechnical site investigation data may not be required |
| **B** As A above but with a thin layer of soft recent sediments over a hard layer of known geology, where the spudcans are expected to be founded on the hard interface beneath the soft recent sediments  
Formation present below the soft / hard interface known to be competent and able to safely support the spudcans | Shallow seabed PCPT(s) may be used to confirm that the absence of potentially adverse layering within the soft upper sediments and to tag the hard interface  
If data prove the soil conditions not as expected then adequate PCPT’s to be conducted investigate and confirm local variability  
If potentially adverse conditions for jack-up foundations are present then consider increasing the geotechnical site investigation work-scope |
<table>
<thead>
<tr>
<th>Site Conditions &amp; Considerations</th>
<th>Geotechnical Site Investigation Recommendations</th>
</tr>
</thead>
<tbody>
<tr>
<td>Regional and local geology well understood</td>
<td>Conduct deep push continuous seabed PCPT boreholes at one spudcan location to the greater of 30m plus the maximum anticipated spudcan penetration depth plus 1.5 spudcan diameters, (provided target depth achievable)</td>
</tr>
<tr>
<td>High quality geophysical survey data available</td>
<td>If deep push PCPT data do not correlate well with desk study and geophysics then conduct further deep push PCPT's at the remaining spudcan locations (provided target depth achievable)</td>
</tr>
<tr>
<td>Soils expected to be laterally continuous</td>
<td>If there are any concerns regarding the suitability of the ground conditions for jack-up operations then conduct additional geotechnical site investigation</td>
</tr>
<tr>
<td>Desk study corroborates geophysical data</td>
<td></td>
</tr>
<tr>
<td>Near-surface conditions expected to be suitable for jack-up operations</td>
<td></td>
</tr>
<tr>
<td>Knowledge of regionally successful jack-up performance and local geotechnical borehole data available</td>
<td></td>
</tr>
<tr>
<td>Unlikely adverse foundation performance risk</td>
<td></td>
</tr>
<tr>
<td>C</td>
<td></td>
</tr>
<tr>
<td>Regional and local geology well understood</td>
<td>If unable to acquire deep push PCPT data to adequate depth conduct a composite (i.e. combined sampling and PCPT) borehole within the jack-up footprint (i.e. between the three leg positions) to the greater of either 30m or the maximum anticipated spudcan penetration plus 1.5 spudcan diameters</td>
</tr>
<tr>
<td>High quality geophysical survey data available without sub-bottom profiling data tie-line(s)</td>
<td>In sand 3 m PCPT, 1 m sample cycle</td>
</tr>
<tr>
<td>Soils expected to be laterally continuous</td>
<td>In clay 3 m PCPT, 2 m sample cycle</td>
</tr>
<tr>
<td>Desk study corroborates geophysical data</td>
<td>Minimum target data gaps &lt; 0.2 m</td>
</tr>
<tr>
<td>Near-surface conditions expected to be suitable for jack-up operations</td>
<td>If data gaps &gt; 0.2 m, or there are concerns regarding the suitability of the ground for jack-up operations then consider additional borehole(s) with downhole PCPT's conducted over data gaps and sampling where PCPT's previously conducted</td>
</tr>
<tr>
<td>Knowledge of regionally successful jack-up performance and local geotechnical borehole data available</td>
<td>If there are any concerns regarding the suitability of the ground conditions for jack-up operations then conduct additional geotechnical site investigation</td>
</tr>
<tr>
<td>Unlikely adverse foundation performance risk</td>
<td></td>
</tr>
</tbody>
</table>
Table D.1 — OPEN LOCATIONS WITHOUT PREVIOUS JACK-UP INSTALLATION

<table>
<thead>
<tr>
<th>Site Conditions &amp; Considerations</th>
<th>Geotechnical Site Investigation Recommendations</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>E</strong> Regional and local geology reasonably well understood with near-surface conditions expected to be continuous and suitable for jack-up operations High quality geophysical survey data available without sub-bottom profiling data tie-line(s) Desk study corroborates geophysical data No local geotechnical data No knowledge of successful jack-up performance regionally Foundation performance risk considered unlikely</td>
<td>Conduct two boreholes, one borehole with continuous sampling and the other with continuous PCPT’s, within the jack-up footprint (i.e. between the three leg positions) to the maximum of either 30m or the anticipated spudcan penetration plus 1,5 spudcan diameters Minimum target data gaps &lt; 0.2 m If data illustrate soil conditions suitable for jack-up operations, and the geophysics confirm stratigraphic continuity across the site then no further investigatory geotechnical works may be necessary Otherwise consider additional geotechnical data acquisition</td>
</tr>
<tr>
<td><strong>F</strong> Regional and local geological data available without specific details of near-surface ground conditions Desk study and geophysical survey data interpretation ambiguous and suggest that the near-surface ground conditions are likely to be variable across the site No knowledge of successful jack-up performance locally (possible jack-up foundation difficulties) No local geotechnical data (possible new soil province) Potential for foundation performance risk recognised</td>
<td>Conduct three continuous downhole PCPT boreholes, one at each spudcan location to the greater of either 30m or the maximum anticipated spudcan penetration plus 1,5 spudcan diameters Where there is good correlation between the PCPT’s conduct a continuous sampling borehole adjacent to one PCPT to facilitate soil testing and PCPT data correlation If data illustrate soil conditions suitable for jack-up operations then no further investigatory geotechnical works may be necessary. Otherwise consider additional data acquisition</td>
</tr>
<tr>
<td><strong>G</strong> As above Where the leg centre PCPT’s illustrate soil variability across the site conduct an additional continuous sampling borehole adjacent and PCPT at the jack-up centre (or an alternative combination of these options) to allow for increased data variation interpretation If data illustrate soil conditions suitable for jack-up operations then no further investigatory geotechnical works may be necessary. Otherwise consider additional data acquisition</td>
<td></td>
</tr>
</tbody>
</table>

(Zone I of Reference [D.1-4])

(Zone II of Reference [D.1-4])

(Zone II of Reference [D.1-4])
### Table D.1 — OPEN LOCATIONS WITHOUT PREVIOUS JACK-UP INSTALLATION

<table>
<thead>
<tr>
<th>Site Conditions &amp; Considerations</th>
<th>Geotechnical Site Investigation Recommendations</th>
</tr>
</thead>
<tbody>
<tr>
<td>H  Regional and local geological data available without specific details of near-surface ground conditions at the site where the desk study and geophysical survey data suggest near-surface ground condition variability across the site (for example complex re-worked infilled channel sequence) Knowledge of adverse jack-up foundation performance locally Limited local geotechnical data (possibly new soil province) High potential for foundation performance risk identified</td>
<td>Conduct three continuous downhole PCPT with adjacent continuous sampling boreholes at each spudcan location to the greater of either 30 m or the maximum anticipated spudcan penetration plus 1.5 spudcan diameters If there is poor correlation between the data at each spudcan location and correlation with the geophysics then conduct additional PCPT and/or sampling boreholes over the jack-up footprint (Zone III of Reference [D.1-4])</td>
</tr>
</tbody>
</table>

Legend:
- Composite Borehole (downhole PCPT and sampling)
- Continuous Sampling Borehole
- Continuous PCPT

### Table D.2 — JACK-UP WORK-OVER LOCATIONS

<table>
<thead>
<tr>
<th>Site Conditions &amp; Considerations</th>
<th>Geotechnical Site Investigation Recommendations</th>
</tr>
</thead>
<tbody>
<tr>
<td>I  First jack-up operation at the location, therefore no spudcan-footprint interaction issues to consider initially – although future implications to be addressed High quality geophysical survey available with recent seabed clearance survey Appropriate geotechnical and geophysical data acquired for fixed platform and jack-up installation purposes confirms suitable ground conditions for jack-up installation Local jack-up operations without foundation hazards</td>
<td>No additional geotechnical data acquired for jack-up installation</td>
</tr>
<tr>
<td>J  Repeat visit – identical jack-up Identical spudcan positions spudcan-footprint interaction issues not possible No previous foundation issues</td>
<td>No new geotechnical data required A recent seabed clearance survey may be required</td>
</tr>
</tbody>
</table>
**Table D.2 — JACK-UP WORK-OVER LOCATIONS**

<table>
<thead>
<tr>
<th>Site Conditions &amp; Considerations</th>
<th>Geotechnical Site Investigation Recommendations</th>
</tr>
</thead>
</table>
| **K**  
Repeat visit – identical jack-up  
New spudcan positions spudcan-footprint interaction issues possible  
Known ground conditions  
No previous foundation issues | Survey of existing footprints advisable with seabed clearance survey  
Consideration of spudcan-footprint interaction mitigation  
Additional geotechnical data may be required |
| **L**  
First visit of jack-up to this platform where unit(s) have previously operated  
New unit with spudcan bearing pressures similar to previous units which have previously operated at the location  
Known ground conditions  
No previous foundation issues | Spudcan-footprint interaction issues to be considered  
Survey of existing footprints advisable with seabed clearance survey  
Consideration of spudcan-footprint interaction mitigation  
Additional geotechnical data may be required |
| **M**  
First visit of jack-up to this platform where unit(s) have previously operated  
New unit with spudcan bearing pressures greater that those of units previously operated at the location | Spudcan-footprint interaction issues to be considered  
Survey of existing footprints advisable with seabed clearance survey  
Consideration of spudcan-footprint interaction mitigation  
Additional geotechnical data may be required. The previous jack-up operations may have modified the ground conditions where at the intended spudcan installation positions and this may require investigation.  
With increased bearing stress the geotechnical conditions to greater depth would have to be considered |
Annex E
(informative)

Foundations: Additional information and alternative approaches
E.1 Guidance on A.9.3.2.3: Penetration in clays – bearing capacity factors of Houlsby and Martin

Presented below are the theoretical solution for the bearing capacity of circular conical foundations on clays of uniform and increasing strength with depth as provided by Houlsby and Martin\[E.1-1\].

In the tables the bearing capacity factors are defined for

- cone angles $\beta$ of 60°, 90°, 120°, 150° and a flat plate of 180°;
- normalised embedment depth ($D/B$) of 0,0; 0,1; 0,25; 0,5; 1,0 and 2,5;
- values of shear strength gradient $\rho B/s_{um}$ between 0 and 5 where $\rho$ is the rate of increase in shear strength with depth, from a value of $s_{um}$ at the sea floor;
- roughness between smooth ($a = 0$) and fully rough ($a = 1$). Roughness is defined as $a = a_u / s_u$ where $a_u$ is the maximum shear stress that can be mobilised at the cone surface and $s_u$ is the local value. Intervals of 0,2 are provided.

Definition of these parameters is provided in Figure E.1-1. The tables provide a theoretical lower bound to the bearing factor $N_{c,s,c}$ to apply to the shear strength at the spudcan base level, $s_{uo}$, for the full range of the above parameters.

NOTE << When re-drawn, replace radius $R$ with diameter $B$ and $c_{ux}$ with $s_{ux} >>

Figure E.1-1 — Conical spudcan bearing capacity - problem definition and notation
Table E.1-1 — Values of $N_{c.s.c.d} = F_V / (a\lambda \rho_{\text{w}})$ for cone angle $\beta = 60^\circ$

<table>
<thead>
<tr>
<th>$\rho B/s_{\text{um}}$</th>
<th>$D/B$</th>
<th>Roughness factor $a = \lambda / s_{\text{u}}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>0,0</td>
<td>0,0</td>
<td>4.45 4.96 5.45 5.90 6.32 6.69</td>
</tr>
<tr>
<td>0,0</td>
<td>0,1</td>
<td>4.68 5.19 5.67 6.12 6.53 6.90</td>
</tr>
<tr>
<td>0,0</td>
<td>0,25</td>
<td>4.98 5.50 5.96 6.40 6.81 7.18</td>
</tr>
<tr>
<td>0,0</td>
<td>0,5</td>
<td>5.41 5.90 6.37 6.81 7.21 7.57</td>
</tr>
<tr>
<td>0,0</td>
<td>1,0</td>
<td>6.07 6.55 7.01 7.43 7.84 8.18</td>
</tr>
<tr>
<td>0,0</td>
<td>2,5</td>
<td>7.33 7.81 8.25 8.66 9.05 9.39</td>
</tr>
<tr>
<td>1,0</td>
<td>0,0</td>
<td>5.81 6.51 7.15 7.77 8.34 8.87</td>
</tr>
<tr>
<td>1,0</td>
<td>0,1</td>
<td>5.92 6.59 7.23 7.83 8.38 8.89</td>
</tr>
<tr>
<td>1,0</td>
<td>0,25</td>
<td>6.04 6.70 7.30 7.88 8.42 8.91</td>
</tr>
<tr>
<td>1,0</td>
<td>0,5</td>
<td>6.20 6.84 7.41 7.96 8.47 8.94</td>
</tr>
<tr>
<td>1,0</td>
<td>1,0</td>
<td>6.43 7.05 7.58 8.12 8.59 9.03</td>
</tr>
<tr>
<td>1,0</td>
<td>2,5</td>
<td>6.97 7.55 8.08 8.54 8.98 9.39</td>
</tr>
<tr>
<td>2,0</td>
<td>0,0</td>
<td>7.14 8.02 8.84 9.60 10.32 10.99</td>
</tr>
<tr>
<td>2,0</td>
<td>0,1</td>
<td>6.92 7.73 8.49 9.21 9.88 10.50</td>
</tr>
<tr>
<td>2,0</td>
<td>0,25</td>
<td>6.74 7.50 8.18 8.84 9.46 10.03</td>
</tr>
<tr>
<td>2,0</td>
<td>0,5</td>
<td>6.59 7.29 7.91 8.53 9.09 9.61</td>
</tr>
<tr>
<td>2,0</td>
<td>1,0</td>
<td>6.55 7.20 7.76 8.33 8.83 9.30</td>
</tr>
<tr>
<td>2,0</td>
<td>2,5</td>
<td>6.99 7.49 8.03 8.50 8.95 9.37</td>
</tr>
<tr>
<td>3,0</td>
<td>0,0</td>
<td>8.49 9.54 10.50 11.42 12.29 13.10</td>
</tr>
<tr>
<td>3,0</td>
<td>0,1</td>
<td>7.77 8.70 9.56 10.38 11.14 11.85</td>
</tr>
<tr>
<td>3,0</td>
<td>0,25</td>
<td>7.24 8.03 8.80 9.53 10.20 10.82</td>
</tr>
<tr>
<td>3,0</td>
<td>0,5</td>
<td>6.82 7.56 8.21 8.86 9.45 10.00</td>
</tr>
<tr>
<td>3,0</td>
<td>1,0</td>
<td>6.60 7.27 7.85 8.44 8.94 9.43</td>
</tr>
<tr>
<td>3,0</td>
<td>2,5</td>
<td>6.99 7.47 8.01 8.49 8.94 9.36</td>
</tr>
<tr>
<td>4,0</td>
<td>0,0</td>
<td>9.83 11.02 12.16 13.24 14.26 15.18</td>
</tr>
<tr>
<td>4,0</td>
<td>0,1</td>
<td>8.51 9.52 10.48 11.38 12.22 13.00</td>
</tr>
<tr>
<td>4,0</td>
<td>0,25</td>
<td>7.61 8.44 9.26 10.04 10.75 11.41</td>
</tr>
<tr>
<td>4,0</td>
<td>0,5</td>
<td>6.97 7.74 8.41 9.08 9.69 10.26</td>
</tr>
<tr>
<td>4,0</td>
<td>1,0</td>
<td>6.64 7.31 7.90 8.49 9.01 9.51</td>
</tr>
<tr>
<td>4,0</td>
<td>2,5</td>
<td>6.86 7.45 8.00 8.48 8.94 9.35</td>
</tr>
<tr>
<td>5,0</td>
<td>0,0</td>
<td>11.17 12.52 13.83 15.06 16.20 17.26</td>
</tr>
<tr>
<td>5,0</td>
<td>0,1</td>
<td>9.14 10.23 11.26 12.25 13.15 13.99</td>
</tr>
<tr>
<td>5,0</td>
<td>0,25</td>
<td>7.90 8.78 9.63 10.43 11.17 11.87</td>
</tr>
<tr>
<td>5,0</td>
<td>0,5</td>
<td>7.08 7.84 8.55 9.24 9.86 10.45</td>
</tr>
<tr>
<td>5,0</td>
<td>1,0</td>
<td>6.66 7.42 7.94 8.53 9.06 9.56</td>
</tr>
<tr>
<td>5,0</td>
<td>2,5</td>
<td>6.85 7.44 7.99 8.47 8.93 9.35</td>
</tr>
</tbody>
</table>
Table E.1-2 — Values of $N_{c,s,c,d_{c}} = F_{V}/(\lambda s_{uo})$ for cone angle $\beta = 90^\circ$

<table>
<thead>
<tr>
<th>$\rho B/s_{um}$</th>
<th>$D/B$</th>
<th>Roughness factor $a = a_{u}/s_{u}$</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>0,0</td>
<td>0,2</td>
</tr>
<tr>
<td>0,0</td>
<td>4,64</td>
<td>5,02</td>
</tr>
<tr>
<td>0,0</td>
<td>4,90</td>
<td>5,28</td>
</tr>
<tr>
<td>0,0</td>
<td>5,22</td>
<td>5,59</td>
</tr>
<tr>
<td>0,0</td>
<td>5,68</td>
<td>6,03</td>
</tr>
<tr>
<td>0,0</td>
<td>6,37</td>
<td>6,71</td>
</tr>
<tr>
<td>0,0</td>
<td>7,65</td>
<td>8,03</td>
</tr>
<tr>
<td>0,0</td>
<td>5,57</td>
<td>6,05</td>
</tr>
<tr>
<td>0,0</td>
<td>5,74</td>
<td>6,21</td>
</tr>
<tr>
<td>0,0</td>
<td>5,94</td>
<td>6,38</td>
</tr>
<tr>
<td>0,0</td>
<td>6,16</td>
<td>6,61</td>
</tr>
<tr>
<td>0,0</td>
<td>6,50</td>
<td>6,93</td>
</tr>
<tr>
<td>0,0</td>
<td>7,25</td>
<td>7,57</td>
</tr>
<tr>
<td>0,0</td>
<td>6,46</td>
<td>7,03</td>
</tr>
<tr>
<td>0,0</td>
<td>6,41</td>
<td>6,94</td>
</tr>
<tr>
<td>0,0</td>
<td>6,41</td>
<td>6,88</td>
</tr>
<tr>
<td>0,0</td>
<td>6,40</td>
<td>6,88</td>
</tr>
<tr>
<td>0,0</td>
<td>6,54</td>
<td>6,99</td>
</tr>
<tr>
<td>0,0</td>
<td>7,12</td>
<td>7,46</td>
</tr>
<tr>
<td>0,0</td>
<td>7,36</td>
<td>8,00</td>
</tr>
<tr>
<td>0,0</td>
<td>6,99</td>
<td>7,57</td>
</tr>
<tr>
<td>0,0</td>
<td>6,70</td>
<td>7,24</td>
</tr>
<tr>
<td>0,0</td>
<td>6,54</td>
<td>7,04</td>
</tr>
<tr>
<td>0,0</td>
<td>6,56</td>
<td>7,02</td>
</tr>
<tr>
<td>0,0</td>
<td>7,12</td>
<td>7,46</td>
</tr>
<tr>
<td>0,0</td>
<td>8,22</td>
<td>8,96</td>
</tr>
<tr>
<td>0,0</td>
<td>7,49</td>
<td>8,11</td>
</tr>
<tr>
<td>0,0</td>
<td>6,94</td>
<td>7,50</td>
</tr>
<tr>
<td>0,0</td>
<td>6,63</td>
<td>7,15</td>
</tr>
<tr>
<td>0,0</td>
<td>6,57</td>
<td>7,03</td>
</tr>
<tr>
<td>0,0</td>
<td>7,05</td>
<td>7,44</td>
</tr>
<tr>
<td>0,0</td>
<td>9,11</td>
<td>9,93</td>
</tr>
<tr>
<td>0,0</td>
<td>7,87</td>
<td>8,55</td>
</tr>
<tr>
<td>0,0</td>
<td>7,12</td>
<td>7,71</td>
</tr>
<tr>
<td>0,0</td>
<td>6,70</td>
<td>7,22</td>
</tr>
<tr>
<td>0,0</td>
<td>6,57</td>
<td>7,04</td>
</tr>
<tr>
<td>0,0</td>
<td>7,03</td>
<td>7,42</td>
</tr>
</tbody>
</table>
Table E.1-3 — Values of $N_{c,s,d_c} = F_V / (As_u)$ for cone angle $\beta = 120^\circ$

<table>
<thead>
<tr>
<th>$\rho B/s_{um}$</th>
<th>$D/B$</th>
<th>Roughness factor $a = a_u / s_u$</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>0,0</td>
<td>0,2</td>
</tr>
<tr>
<td>0,0</td>
<td>4,96</td>
<td>5,25</td>
</tr>
<tr>
<td>0,0</td>
<td>5,23</td>
<td>5,52</td>
</tr>
<tr>
<td>0,0</td>
<td>5,57</td>
<td>5,85</td>
</tr>
<tr>
<td>0,0</td>
<td>6,04</td>
<td>6,31</td>
</tr>
<tr>
<td>0,0</td>
<td>6,74</td>
<td>7,01</td>
</tr>
<tr>
<td>0,0</td>
<td>8,07</td>
<td>8,32</td>
</tr>
<tr>
<td>1,0</td>
<td>5,69</td>
<td>6,04</td>
</tr>
<tr>
<td>1,0</td>
<td>5,99</td>
<td>6,24</td>
</tr>
<tr>
<td>1,0</td>
<td>6,12</td>
<td>6,45</td>
</tr>
<tr>
<td>1,0</td>
<td>6,39</td>
<td>6,72</td>
</tr>
<tr>
<td>1,0</td>
<td>6,80</td>
<td>7,10</td>
</tr>
<tr>
<td>1,0</td>
<td>7,52</td>
<td>7,82</td>
</tr>
<tr>
<td>2,0</td>
<td>6,38</td>
<td>6,79</td>
</tr>
<tr>
<td>2,0</td>
<td>6,41</td>
<td>6,80</td>
</tr>
<tr>
<td>2,0</td>
<td>6,46</td>
<td>6,83</td>
</tr>
<tr>
<td>2,0</td>
<td>6,56</td>
<td>6,91</td>
</tr>
<tr>
<td>2,0</td>
<td>6,80</td>
<td>7,12</td>
</tr>
<tr>
<td>2,0</td>
<td>7,43</td>
<td>7,72</td>
</tr>
<tr>
<td>3,0</td>
<td>7,04</td>
<td>7,51</td>
</tr>
<tr>
<td>3,0</td>
<td>6,84</td>
<td>7,27</td>
</tr>
<tr>
<td>3,0</td>
<td>6,71</td>
<td>7,09</td>
</tr>
<tr>
<td>3,0</td>
<td>6,66</td>
<td>7,02</td>
</tr>
<tr>
<td>3,0</td>
<td>6,81</td>
<td>7,11</td>
</tr>
<tr>
<td>3,0</td>
<td>7,38</td>
<td>7,68</td>
</tr>
<tr>
<td>4,0</td>
<td>7,70</td>
<td>8,22</td>
</tr>
<tr>
<td>4,0</td>
<td>7,20</td>
<td>7,66</td>
</tr>
<tr>
<td>4,0</td>
<td>6,88</td>
<td>7,28</td>
</tr>
<tr>
<td>4,0</td>
<td>6,72</td>
<td>7,08</td>
</tr>
<tr>
<td>4,0</td>
<td>6,80</td>
<td>7,12</td>
</tr>
<tr>
<td>4,0</td>
<td>7,39</td>
<td>7,66</td>
</tr>
<tr>
<td>5,0</td>
<td>8,35</td>
<td>8,91</td>
</tr>
<tr>
<td>5,0</td>
<td>7,52</td>
<td>7,99</td>
</tr>
<tr>
<td>5,0</td>
<td>7,01</td>
<td>7,43</td>
</tr>
<tr>
<td>5,0</td>
<td>6,77</td>
<td>7,13</td>
</tr>
<tr>
<td>5,0</td>
<td>6,80</td>
<td>7,12</td>
</tr>
<tr>
<td>5,0</td>
<td>7,34</td>
<td>7,64</td>
</tr>
</tbody>
</table>
Table E.1-4 — Values of \( N_{c,s,c} d_c = F_V / \left( A s_u \right) \) for cone angle \( \beta = 150^\circ \)

<table>
<thead>
<tr>
<th>( \rho B/s_{um} )</th>
<th>( D/B )</th>
<th>Roughness factor ( a = a_u / s_u )</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>0,0</td>
<td>0,2</td>
</tr>
<tr>
<td>0,0</td>
<td>5,32</td>
<td>5,55</td>
</tr>
<tr>
<td>0,1</td>
<td>5,60</td>
<td>5,82</td>
</tr>
<tr>
<td>0,25</td>
<td>5,94</td>
<td>6,16</td>
</tr>
<tr>
<td>0,5</td>
<td>6,41</td>
<td>6,62</td>
</tr>
<tr>
<td>1,0</td>
<td>7,13</td>
<td>7,32</td>
</tr>
<tr>
<td>2,5</td>
<td>8,46</td>
<td>8,65</td>
</tr>
<tr>
<td>1,0  0,0</td>
<td>5,94</td>
<td>6,22</td>
</tr>
<tr>
<td>1,0  0,1</td>
<td>6,16</td>
<td>6,43</td>
</tr>
<tr>
<td>1,0  0,25</td>
<td>6,41</td>
<td>6,67</td>
</tr>
<tr>
<td>1,0  0,5</td>
<td>6,71</td>
<td>6,96</td>
</tr>
<tr>
<td>1,0  1,0</td>
<td>7,13</td>
<td>7,36</td>
</tr>
<tr>
<td>1,0  2,5</td>
<td>7,91</td>
<td>8,12</td>
</tr>
<tr>
<td>2,0  0,0</td>
<td>6,50</td>
<td>6,82</td>
</tr>
<tr>
<td>2,0  0,1</td>
<td>6,59</td>
<td>6,90</td>
</tr>
<tr>
<td>2,0  0,25</td>
<td>6,69</td>
<td>6,98</td>
</tr>
<tr>
<td>2,0  0,5</td>
<td>6,84</td>
<td>7,10</td>
</tr>
<tr>
<td>2,0  1,0</td>
<td>7,11</td>
<td>7,35</td>
</tr>
<tr>
<td>2,0  2,5</td>
<td>7,81</td>
<td>8,01</td>
</tr>
<tr>
<td>3,0  0,0</td>
<td>7,03</td>
<td>7,40</td>
</tr>
<tr>
<td>3,0  0,1</td>
<td>6,94</td>
<td>7,27</td>
</tr>
<tr>
<td>3,0  0,25</td>
<td>6,88</td>
<td>7,18</td>
</tr>
<tr>
<td>3,0  0,5</td>
<td>6,91</td>
<td>7,18</td>
</tr>
<tr>
<td>3,0  1,0</td>
<td>7,10</td>
<td>7,35</td>
</tr>
<tr>
<td>3,0  2,5</td>
<td>7,76</td>
<td>7,97</td>
</tr>
<tr>
<td>4,0  0,0</td>
<td>7,55</td>
<td>7,94</td>
</tr>
<tr>
<td>4,0  0,1</td>
<td>7,23</td>
<td>7,58</td>
</tr>
<tr>
<td>4,0  0,25</td>
<td>7,02</td>
<td>7,34</td>
</tr>
<tr>
<td>4,0  0,5</td>
<td>6,95</td>
<td>7,23</td>
</tr>
<tr>
<td>4,0  1,0</td>
<td>7,09</td>
<td>7,34</td>
</tr>
<tr>
<td>4,0  2,5</td>
<td>7,72</td>
<td>7,94</td>
</tr>
<tr>
<td>5,0  0,0</td>
<td>8,05</td>
<td>8,48</td>
</tr>
<tr>
<td>5,0  0,1</td>
<td>7,46</td>
<td>7,83</td>
</tr>
<tr>
<td>5,0  0,25</td>
<td>7,13</td>
<td>7,45</td>
</tr>
<tr>
<td>5,0  0,5</td>
<td>6,99</td>
<td>7,27</td>
</tr>
<tr>
<td>5,0  1,0</td>
<td>7,09</td>
<td>7,34</td>
</tr>
<tr>
<td>5,0  2,5</td>
<td>7,70</td>
<td>7,93</td>
</tr>
</tbody>
</table>
Table E.1-5  —  Values of \( N_{c,c_d,c_s} = F_V / (\Delta s_{\text{um}}) \) for cone angle \( \beta = 180^\circ \)

<table>
<thead>
<tr>
<th>( \rho B/s_{\text{um}} )</th>
<th>D/B</th>
<th>Roughness factor ( a = a_u / s_u )</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>0.0</td>
</tr>
<tr>
<td>0.0</td>
<td>0.0</td>
<td>5.69</td>
</tr>
<tr>
<td>0.0</td>
<td>0.1</td>
<td>5.97</td>
</tr>
<tr>
<td>0.0</td>
<td>0.25</td>
<td>6.31</td>
</tr>
<tr>
<td>0.0</td>
<td>0.5</td>
<td>6.79</td>
</tr>
<tr>
<td>0.0</td>
<td>1.0</td>
<td>7.49</td>
</tr>
<tr>
<td>0.0</td>
<td>2.5</td>
<td>8.82</td>
</tr>
<tr>
<td>1.0</td>
<td>0.0</td>
<td>6.25</td>
</tr>
<tr>
<td>1.0</td>
<td>0.1</td>
<td>6.48</td>
</tr>
<tr>
<td>1.0</td>
<td>0.25</td>
<td>6.74</td>
</tr>
<tr>
<td>1.0</td>
<td>0.5</td>
<td>7.05</td>
</tr>
<tr>
<td>1.0</td>
<td>1.0</td>
<td>7.47</td>
</tr>
<tr>
<td>1.0</td>
<td>2.5</td>
<td>8.26</td>
</tr>
<tr>
<td>2.0</td>
<td>0.0</td>
<td>6.73</td>
</tr>
<tr>
<td>2.0</td>
<td>0.1</td>
<td>6.85</td>
</tr>
<tr>
<td>2.0</td>
<td>0.25</td>
<td>6.98</td>
</tr>
<tr>
<td>2.0</td>
<td>0.5</td>
<td>7.15</td>
</tr>
<tr>
<td>2.0</td>
<td>1.0</td>
<td>7.45</td>
</tr>
<tr>
<td>2.0</td>
<td>2.5</td>
<td>8.16</td>
</tr>
<tr>
<td>3.0</td>
<td>0.0</td>
<td>7.16</td>
</tr>
<tr>
<td>3.0</td>
<td>0.1</td>
<td>7.13</td>
</tr>
<tr>
<td>3.0</td>
<td>0.25</td>
<td>7.15</td>
</tr>
<tr>
<td>3.0</td>
<td>0.5</td>
<td>7.21</td>
</tr>
<tr>
<td>3.0</td>
<td>1.0</td>
<td>7.43</td>
</tr>
<tr>
<td>3.0</td>
<td>2.5</td>
<td>8.13</td>
</tr>
<tr>
<td>4.0</td>
<td>0.0</td>
<td>7.56</td>
</tr>
<tr>
<td>4.0</td>
<td>0.1</td>
<td>7.38</td>
</tr>
<tr>
<td>4.0</td>
<td>0.25</td>
<td>7.26</td>
</tr>
<tr>
<td>4.0</td>
<td>0.5</td>
<td>7.25</td>
</tr>
<tr>
<td>4.0</td>
<td>1.0</td>
<td>7.44</td>
</tr>
<tr>
<td>4.0</td>
<td>2.5</td>
<td>8.09</td>
</tr>
<tr>
<td>5.0</td>
<td>0.0</td>
<td>7.94</td>
</tr>
<tr>
<td>5.0</td>
<td>0.1</td>
<td>7.56</td>
</tr>
<tr>
<td>5.0</td>
<td>0.25</td>
<td>7.34</td>
</tr>
<tr>
<td>5.0</td>
<td>0.5</td>
<td>7.27</td>
</tr>
<tr>
<td>5.0</td>
<td>1.0</td>
<td>7.43</td>
</tr>
<tr>
<td>5.0</td>
<td>2.5</td>
<td>8.07</td>
</tr>
</tbody>
</table>
E.2 Guidance on A.9.3.2.4: Penetration in silica sands

Theoretical values of $N_γ$ calculated using the slip-line method for circular footings with a conical underside [E.2-1] (Cassidy and Houlsby, 2002) are given in Tables E.2-1 to E.2-6 (see also Figure E.2-1). These values cover cone apex angles from 60° to 180°, spudcan-soil interface roughness coefficients ($α = \tan δ′ / \tan φ′$) from 0,6 to 1, and soil friction angles from 20° to 40°. The soil–steel interface friction angle, $δ′$, is typically in the range 22° - 29°, decreasing with increasing grain size API [E.2-2]. This implies that the roughness coefficient $α$ is at least 0,6 for typical soil friction angles ($φ′ = 30° \pm 5°$). Note that the $N_γ$ values given in Table A.9.3-3 for the special case of a flat, rough circular footing (i.e. a 180° cone with $α = 1$) were calculated using a different implementation of the slip-line method, Martin [E.2-1].

![Figure E.2-1 – Definition of parameters for Tables E.2-1 to E.2-5](image)

<table>
<thead>
<tr>
<th>$\beta = 60°$</th>
<th>$\phi (°)$</th>
<th>$α = 1$</th>
<th>$α = 0,8$</th>
<th>$α = 0,6$</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>20</td>
<td>7,33</td>
<td>6,55</td>
<td>5,64</td>
</tr>
<tr>
<td></td>
<td>25</td>
<td>14,69</td>
<td>12,99</td>
<td>10,94</td>
</tr>
<tr>
<td></td>
<td>30</td>
<td>31,99</td>
<td>27,45</td>
<td>22,50</td>
</tr>
<tr>
<td></td>
<td>35</td>
<td>79,26</td>
<td>62,95</td>
<td>48,81</td>
</tr>
<tr>
<td></td>
<td>40</td>
<td>209,2</td>
<td>163,2</td>
<td>122,3</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>$\beta = 90°$</th>
<th>$\phi (°)$</th>
<th>$α = 1$</th>
<th>$α = 0,8$</th>
<th>$α = 0,6$</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>20</td>
<td>4,54</td>
<td>4,11</td>
<td>3,62</td>
</tr>
<tr>
<td></td>
<td>25</td>
<td>9,58</td>
<td>8,50</td>
<td>7,26</td>
</tr>
<tr>
<td></td>
<td>30</td>
<td>21,12</td>
<td>18,87</td>
<td>15,58</td>
</tr>
<tr>
<td></td>
<td>35</td>
<td>51,76</td>
<td>47,42</td>
<td>37,01</td>
</tr>
<tr>
<td></td>
<td>40</td>
<td>142,8</td>
<td>132,6</td>
<td>99,18</td>
</tr>
</tbody>
</table>

Table E.2-1 — Bearing capacity factors ($N_γ$) for a conical apex angle of 60°

Table E.2-2 — Bearing capacity factors ($N_γ$) for a conical apex angle of 90°
Table E.2-3 — Bearing capacity factors ($N_γ$) for a conical apex angle of 120°

<table>
<thead>
<tr>
<th>$\phi$ (°)</th>
<th>$\alpha = 1$</th>
<th>$\alpha = 0.8$</th>
<th>$\alpha = 0.6$</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\beta = 120°$</td>
<td>$\alpha = 1$</td>
<td>$\alpha = 0.8$</td>
<td>$\alpha = 0.6$</td>
</tr>
<tr>
<td>20</td>
<td>3.37</td>
<td>3.15</td>
<td>2.81</td>
</tr>
<tr>
<td>25</td>
<td>7.46</td>
<td>6.99</td>
<td>6.05</td>
</tr>
<tr>
<td>30</td>
<td>17.58</td>
<td>16.80</td>
<td>14.30</td>
</tr>
<tr>
<td>35</td>
<td>44.73</td>
<td>42.99</td>
<td>35.79</td>
</tr>
<tr>
<td>40</td>
<td>129.4</td>
<td>124.9</td>
<td>103.3</td>
</tr>
</tbody>
</table>

Table E.2-4 — Bearing capacity factors ($N_γ$) for a conical apex angle of 150°

<table>
<thead>
<tr>
<th>$\phi$ (°)</th>
<th>$\alpha = 1$</th>
<th>$\alpha = 0.8$</th>
<th>$\alpha = 0.6$</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\beta = 150°$</td>
<td>$\alpha = 1$</td>
<td>$\alpha = 0.8$</td>
<td>$\alpha = 0.6$</td>
</tr>
<tr>
<td>20</td>
<td>2.72</td>
<td>2.61</td>
<td>2.39</td>
</tr>
<tr>
<td>25</td>
<td>6.44</td>
<td>6.13</td>
<td>5.60</td>
</tr>
<tr>
<td>30</td>
<td>15.93</td>
<td>15.13</td>
<td>14.02</td>
</tr>
<tr>
<td>35</td>
<td>42.36</td>
<td>40.42</td>
<td>36.41</td>
</tr>
<tr>
<td>40</td>
<td>128.1</td>
<td>120.5</td>
<td>110.5</td>
</tr>
</tbody>
</table>

Table E.2-5 — Bearing capacity factors ($N_γ$) for a conical apex angle of 180°

<table>
<thead>
<tr>
<th>$\phi$ (°)</th>
<th>$\alpha = 1$</th>
<th>$\alpha = 0.8$</th>
<th>$\alpha = 0.6$</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\beta = 180°$</td>
<td>$\alpha = 1$</td>
<td>$\alpha = 0.8$</td>
<td>$\alpha = 0.6$</td>
</tr>
<tr>
<td>20</td>
<td>2.16</td>
<td>2.04</td>
<td>1.99</td>
</tr>
<tr>
<td>25</td>
<td>5.27</td>
<td>5.37</td>
<td>5.14</td>
</tr>
<tr>
<td>30</td>
<td>14.13</td>
<td>13.91</td>
<td>12.98</td>
</tr>
<tr>
<td>35</td>
<td>42.56</td>
<td>40.93</td>
<td>36.81</td>
</tr>
<tr>
<td>40</td>
<td>129.4</td>
<td>121.5</td>
<td>117.0</td>
</tr>
</tbody>
</table>

The bearing capacity calculated using Equation (9.3-5) is strongly dependent on the adopted soil friction angle, $\phi'$. The apparent friction angle mobilised during spudcan penetration in sand is influenced by:

1. The soil relative density (and therefore the dilatancy) – peak friction angle increases with relative density
2. The size of the spudcan, and therefore the stress level within the failing soil – peak friction angle reduces as the stress level increases
3. Progressive failure – soil elements at different locations within the failure mechanism have undergone widely differing levels of shear strain
4. Progressive failure due to pre-shearing of the soil by the conical spudcan tip – which acts to reduce the mobilised peak strength
5. Compression of the foundation soil – which generates additional settlement
6. The level of drainage (i.e. excess pore pressure development) – which changes the effective stress and therefore the soil strength

The soil friction angle can be assessed from laboratory tests, such as triaxial compression tests. To obtain the appropriate response, these tests should be carried out on samples at the relevant relative density and stress level, due to effects (1) and (2) above. Many procedures have been proposed for selecting a representative stress level between the in situ stress and the (average) foundation bearing pressure; a stress of \(\sim 10\%\) of the bearing pressure is typically found to be appropriate (see e.g. Perkins and Madson\(^{\text{E.2-2}}\), Randolph et al.\(^{\text{E.2-3}}\), White et al.\(^{\text{E.2-4}}\)). Alternatively, correlations with CPT parameters can be used to assess the spudcan penetration directly according to Schmertmann\(^{\text{E.2-5}}\), or to infer the soil relative density, from which the peak friction angle can be estimated.

However, the apparent friction angle mobilised during spudcan penetration is lower than the peak value measured in the laboratory (or inferred using CPT correlations), due to mechanisms (3) to (5) above. Back-analyses of field penetration records Cassidy et al.\(^{\text{E.2-6}}\) and centrifuge tests White et al.\(^{\text{E.2-7}}\) have indicated that the friction angle is similar to the critical state friction angle, increasing by up to 5° with increasing relative density.

If preloading is conducted too quickly for drained conditions to prevail, then positive excess pore pressures can be generated beneath the spudcan, leading to a reduction in bearing capacity (mechanism (6) above). This possibility is particularly relevant for skirted spudcans.

E.3 Guidance on A.9.3.2.7.4: Punch-through on: Sand overlying clay: further details on alternate methods

Teh et al. (2008b) investigated the change in bearing failure mechanism during spudcan continuous penetration in sand overlying clay. This led to an alternative approach of evaluating the spudcan penetration resistance-depth \(q_{\text{pen}}-d\) profile based on a simplified profile shown in Figure E.3-1. The spudcan bearing resistance can hence be represented by the ultimate bearing capacity at original ground surface (i.e. \(d = 0\)), \(q_0\), followed by the peak ultimate bearing capacity, \(q_{\text{peak}}\) (at \(d = d_{\text{crit}}\)), and finally the ultimate spudcan bearing capacity when penetrating into the underlying clay (for \(d \geq H\)). Hence \(d_{\text{crit}}\) refers to the depth where punch through occurs and \(H\) is the thickness of the upper sand layer.
The formulation for estimating the three characteristic ultimate bearing capacities are given in Teh et al.\cite{E.3-1} and summarised as follows.

a) When the spudcan widest cross-section is at the original ground surface (at $d = 0$), the failure mechanism shown in Figure E.3-2 (a) shows that the ultimate bearing capacity $q_0$ consists of shearing developing along vertical planes in the overlying sand layer; and general bearing shear failure in the underlying clay. Consideration of soil backflow is not applicable.

b) The failure mechanism at the instant of punch through illustrated in Figure E.3-2 (b) reveals that $q_{peak}$ consists of shearing along logarithmic spiral failure planes in the upper sand layer; and the mobilisation of underlying clay bearing capacity subjected to vertical and inclined loadings. The inclusion of the inclined loading in the assessment of the underlying clay bearing capacity is made in view of the presence of shear stress at the clay surface (Love et al.\cite{E.3-2}; Burd & Fryman\cite{E.3-3}; Teh et al.\cite{E.3-4}). Soil backflow is minimal and can hence be ignored.

c) When the spudcan penetrates through the underlying sand layer, the failure mechanism shown in Figure E.3-2 (c) shows that $q_{nom}$ for $d \geq H$ can be assessed by considering the resistance of a sand plug trapped underneath the spudcan with additional side friction (Craig & Chua\cite{E.3-5}) and a new design depth at the sand plug base elevation. Deep flow mechanism is assumed to occur around the sand plug base. Complete soil backflow is considered here.
Figure E.3-2 — Schematic of spudcan failure mechanisms
Figures E.3-3 and E.3-4 compare the ratio of calculated over measured $q_0$ and $q_{\text{peak}}$, respectively. The calculated values were produced by various methods as indicated in the figures; whereas the measured values were obtained from 18 centrifuge tests reported by Teh et al.\cite{E.3-6} and Craig & Chua\cite{E.3-5}. The above verification is based on limited test data and further studies may be necessary to evaluate the above proposed approach in the determination of spudcan punch through in sand overlying clay.

**Figure E.3-3 — Summary of $q_0,\text{calculated}/q_0,\text{measured}$ ratio given by various methods**
Figure E.3-4 — Summary of $q_{\text{peak,calculated}}/q_{\text{peak,measured}}$ ratio given by various methods

Examining the chart, one can observe the variability in the ratios calculated by different methods. The ratios are compared against each other, along with their respective standard deviations (S.D.) and the data source, which is indicated as 18 centrifuge tests. For instance, the punching shear method by Hanna & Meyerhof (1980) shows a mean ratio of approximately 0.27 with an S.D. of 0.27, while the projected area method by Teh et al. (2008c) exhibits a mean ratio of 1.9 with an S.D. of 0.18.
Annex F
(informative)

Informative annex on A.12: Structural strength
NOTE << In this Annex, the minor axis should be the Z-axis, not the X-axis; subscripts are to be changed accordingly. >>

F.1 Guidance on A.12.6.2.4: Axial compressive column buckling strength

NOTE << It is anticipated that the text of F.1 will be updated/refined (by Dave Lewis). >>

The formulations in AISC 2005\textsuperscript{[A.12.5-1]} are supposedly based on an approximation to the SSRC Curve 2P. In fact, the Ref. AISC column curve, although a reasonable approximation to Curve 2P for $\lambda < 1.5$ and $\lambda > 1.8$, actually lies closer to SSRC Curve 1P for $\lambda = 1.3$ than it does to Curve 2P. Notwithstanding, Curve 2P and thus the AISC curve relate primarily to traditional building construction steels typically with yield stresses up to 345 N/mm\textsuperscript{2} together with their corresponding column tolerances of length/1000. However, because of the high strength steel used in the construction of chords in particular, up to 700 N/mm\textsuperscript{2}, and possibly tighter column tolerances at around length/1500, it is likely that a higher SSRC Column Curve is appropriate for the assessment of chord members. The relevance of yield stress is that the effects of welding and rolling residual stresses reduce as the yield stress of the material increases.

SSRC Column Curve 1P was investigated as a possible candidate curve. A good approximation to this, within 0.8\% of the SSRC expression from $\lambda = 0.0$ to 2.0, is as shown above.

Preliminary attempts have been made to see if it is possible to relatively simply allocate typical chord sections to this 1P curve or to the Ref. A.5 curve. However, this process has not been successful primarily because there are no data (test or numerical) for chord or other high strength steel sections on which to base such allocation. Nevertheless, should a jack-up leg fabricator be able to demonstrate that a chord section consistently achieves the strengths consistent with those of SSRC Curve 1P, then (A.12.7-1) may be used in the assessment of such chords.

Comparison of Column Curves
F.2 Guidance on A.12.6.3.2: The interaction equation approach - determination of $\eta$

Determination of the correct value of $\eta$ is carried out by calculation of the nominal strength of the member about axes other than the x- and y-axes. This can be done in the normal manner based on the effective plastic section modulus with reductions for local buckling if applicable. Although a beam will not necessarily bend in the same plane as the applied moment when the bending plane is at an angle to the orthogonal axes, it is not expected that the capacity will be greatly affected.

Once the nominal bending strength has been calculated for a few angles between the x- and y-axes, the value for $\eta$ can be calculated using a graphical procedure, or by an iterative procedure. A successful iterative procedure has been found to be by the use of the coupled equations, setting $a = M_{ux}' / M_{ux}$ and $b = M_{uy}' / M_{uy}$:

$$\eta_{i+1} = \frac{\ln(1-b\eta)}{\ln(a)}$$  \hspace{1cm} (F.2-1)

with the accelerating step:

$$\eta_{i+2} = 0.5(\eta_{i+1} + \eta_i)$$  \hspace{1cm} (F.2-2)

and the initial value $\eta = 1.5$.

The three angles chosen, 30°, 45° and 60° give a good spread over the 90° range. It is not the intention to fit a curve through all the values from the three angles but merely find the lowest value to $\eta$. This can still make the equation conservative although considerably less so than for $\eta = 1.0$. 

© ISO 2009 – All rights reserved
Figure F.2-1 — Graphical approach to the determination of $\eta$

F.3 Guidance on A.12.6.3.3: The interaction surface approach

The following interaction surfaces and data are based on Dyer, Reference [F.2-1].

Strength interaction equations

\[
\left( \frac{M_x}{M'_{px}} \right)^2 + \left( \frac{M_y}{M'_{py}} \right)^2 \leq 1.00
\]

For \( P/P_y \leq 0.6 \):

\[
M'_{px} = M_{px} \left[ \cos \left( \frac{\pi P}{2P_y} \right) \right]^{0.7}
\]

\[
M'_{py} = M_{py} \left[ \cos \left( \frac{\pi P}{2P_y} \right) \right]^{1.1}
\]

For \( P/P_y > 0.6 \):

\[
M'_{px} = 1.71 M_{px} \left[ 1 - \frac{P}{P_y} \right]
\]

\[
M'_{py} = 1.39 M_{py} \left[ 1 - \frac{P}{P_y} \right]
\]

Figure F.3-1 — Interaction equations/curves for tubular chords with double central racks.
Table F.3-1 — Data for tubular chords with double central racks

Legend: Chord dimensions:

<table>
<thead>
<tr>
<th>Design</th>
<th>L1</th>
<th>t1</th>
<th>L2</th>
<th>t2</th>
<th>D</th>
<th>t3</th>
<th>Fy1</th>
<th>Fy2</th>
<th>Fy3</th>
<th>Bay Ht</th>
</tr>
</thead>
<tbody>
<tr>
<td>BMC JU-300-CAN (Zapata Scotian)</td>
<td>991</td>
<td>127</td>
<td>0</td>
<td>0</td>
<td>914</td>
<td>44</td>
<td>690</td>
<td>0</td>
<td>690</td>
<td>5532</td>
</tr>
<tr>
<td>CFEM T2001 (Hitachi Redesign)</td>
<td>960</td>
<td>18</td>
<td>121</td>
<td>140</td>
<td>960</td>
<td>52</td>
<td>690</td>
<td>690</td>
<td>690</td>
<td>4500 Btm 3 bays</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>34</td>
<td>690</td>
<td>690</td>
<td>690</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>26</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>CFEM T2005</td>
<td>650</td>
<td>20</td>
<td>108</td>
<td>140</td>
<td>800</td>
<td>30</td>
<td>700</td>
<td>685</td>
<td>650</td>
<td>650 or 5050</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>31</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>32</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>33</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>35</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>36</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>38</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>40</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>44</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>34</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>38</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>42</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

* Early CFEM T2005 designs use 650 MPa steel for tube, later designs use 700 MPa steel.

...continued
Table F.3-1 — (continued) Data for tubular chords with double central racks

<table>
<thead>
<tr>
<th>Design</th>
<th>L1</th>
<th>t1</th>
<th>L2</th>
<th>t2</th>
<th>D</th>
<th>t3</th>
<th>Fy1</th>
<th>Fy2</th>
<th>Fy3</th>
<th>Bay Ht</th>
</tr>
</thead>
<tbody>
<tr>
<td>CFEM T2600</td>
<td>650</td>
<td>20</td>
<td>120</td>
<td>140</td>
<td>800</td>
<td>33</td>
<td>700</td>
<td>700</td>
<td>700</td>
<td>6000</td>
</tr>
<tr>
<td>MODEC 200</td>
<td>450</td>
<td>15</td>
<td>102</td>
<td>127</td>
<td>559</td>
<td>27</td>
<td>490</td>
<td>690</td>
<td>490</td>
<td>5486</td>
</tr>
<tr>
<td>MODEC 300</td>
<td>450</td>
<td>25</td>
<td>102</td>
<td>127</td>
<td>559</td>
<td>34</td>
<td>490</td>
<td>690</td>
<td>490</td>
<td>5486</td>
</tr>
<tr>
<td>MODEC 400 (Trident 9)</td>
<td>690</td>
<td>20</td>
<td>102</td>
<td>127</td>
<td>800</td>
<td>30</td>
<td>490</td>
<td>690</td>
<td>490</td>
<td>6200</td>
</tr>
<tr>
<td>Hitachi K1025/31/32</td>
<td>900</td>
<td>18</td>
<td>100</td>
<td>127</td>
<td>900</td>
<td>32</td>
<td>690</td>
<td>690</td>
<td>690</td>
<td>5160</td>
</tr>
<tr>
<td>Hitachi K1026 (Neddrill 4)</td>
<td>950</td>
<td>18</td>
<td>100</td>
<td>127</td>
<td>950</td>
<td>32</td>
<td>690</td>
<td>690</td>
<td>690</td>
<td>4360</td>
</tr>
<tr>
<td>Hitachi K1056/7</td>
<td>1000</td>
<td>28</td>
<td>130</td>
<td>178</td>
<td>1000</td>
<td>47</td>
<td>690</td>
<td>730</td>
<td>690</td>
<td>4600</td>
</tr>
<tr>
<td>ETA Robray 300 (Asia Class)</td>
<td>627</td>
<td>10</td>
<td>127</td>
<td>127</td>
<td>762</td>
<td>22</td>
<td>690</td>
<td>690</td>
<td>690</td>
<td>5486</td>
</tr>
<tr>
<td>ETA Europe Class</td>
<td>627</td>
<td>38</td>
<td>140</td>
<td>140</td>
<td>762</td>
<td>22</td>
<td>690</td>
<td>690</td>
<td>690</td>
<td>5486</td>
</tr>
</tbody>
</table>
Strength interaction equations:

\[
\left( \frac{M_x}{M'_{px}} \right)^2 + \left( \frac{M_y}{M'_{py}} \right)^2 \leq 1.00
\]

where:

\[
M'_{px} = M_{px} \left( 1 - \left( \frac{P}{P_y} \right)^{2.25} \right)
\]

\[
M'_{py} = M_{py} \left( 1 - \left( \frac{P}{P_y} \right)^{1.85} \right)
\]

Figure F.3-2 — Interaction equations/curves for split tubular chords with opposed central racks (doubly symmetrical)
Table F.3-2 — Data for split tubular chords with double central racks

Legend: Chord dimensions

<table>
<thead>
<tr>
<th>Design</th>
<th>L1</th>
<th>t1</th>
<th>D</th>
<th>t2</th>
<th>t3</th>
<th>L4</th>
<th>H1</th>
<th>H2</th>
<th>Fy1</th>
<th>Ht</th>
</tr>
</thead>
<tbody>
<tr>
<td>F &amp; G L780 (Lower bays)</td>
<td>400</td>
<td>152</td>
<td>381</td>
<td>25</td>
<td>25</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>191</td>
<td>621</td>
</tr>
<tr>
<td>F &amp; G L780 (Upper bays)</td>
<td>400</td>
<td>127</td>
<td>381</td>
<td>25</td>
<td>25</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>191</td>
<td>621</td>
</tr>
<tr>
<td>F &amp; G L780 m2 (Lower bays)</td>
<td>400</td>
<td>152</td>
<td>381</td>
<td>32</td>
<td>32</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>191</td>
<td>621</td>
</tr>
<tr>
<td>F &amp; G L780 m2 (Upper bays)</td>
<td>400</td>
<td>127</td>
<td>381</td>
<td>32</td>
<td>32</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>191</td>
<td>621</td>
</tr>
<tr>
<td>F &amp; G L780 m5 (Monitor)</td>
<td>401</td>
<td>178</td>
<td>381</td>
<td>32</td>
<td>32</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>178</td>
<td>621</td>
</tr>
<tr>
<td>F &amp; G L780 m5 (Monarch)</td>
<td>401</td>
<td>178</td>
<td>381</td>
<td>32</td>
<td>32</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>178</td>
<td>621</td>
</tr>
<tr>
<td>F &amp; G L780 m6</td>
<td>611</td>
<td>178</td>
<td>584</td>
<td>83</td>
<td>83</td>
<td>0</td>
<td>0</td>
<td>95</td>
<td>292</td>
<td>621</td>
</tr>
<tr>
<td>MSC CJ62 (Lower bays)</td>
<td>650</td>
<td>210</td>
<td>600</td>
<td>65</td>
<td>48</td>
<td>0</td>
<td>0</td>
<td>75</td>
<td>270</td>
<td>621</td>
</tr>
<tr>
<td>MSC CJ62 (Upper bays)</td>
<td>650</td>
<td>210</td>
<td>600</td>
<td>55</td>
<td>40</td>
<td>0</td>
<td>0</td>
<td>75</td>
<td>270</td>
<td>621</td>
</tr>
<tr>
<td>MSC CJ50 (1)</td>
<td>550</td>
<td>210</td>
<td>520</td>
<td>25</td>
<td>25</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>260</td>
<td>621</td>
</tr>
<tr>
<td>MSC CJ50 (2)</td>
<td>550</td>
<td>210</td>
<td>520</td>
<td>25</td>
<td>25</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>260</td>
<td>621</td>
</tr>
<tr>
<td>Technip TPG 500 (1)</td>
<td>722</td>
<td>160</td>
<td>680</td>
<td>75</td>
<td>61</td>
<td>0</td>
<td>0</td>
<td>20</td>
<td>340</td>
<td>621</td>
</tr>
<tr>
<td>Technip TPG 500 (2)</td>
<td>722</td>
<td>160</td>
<td>680</td>
<td>75</td>
<td>37</td>
<td>0</td>
<td>0</td>
<td>55</td>
<td>340</td>
<td>621</td>
</tr>
<tr>
<td>Technip TPG 500 (3)</td>
<td>722</td>
<td>160</td>
<td>680</td>
<td>62</td>
<td>37</td>
<td>0</td>
<td>0</td>
<td>36</td>
<td>340</td>
<td>621</td>
</tr>
<tr>
<td>Technip TPG 500 (4)</td>
<td>722</td>
<td>160</td>
<td>680</td>
<td>58</td>
<td>37</td>
<td>0</td>
<td>0</td>
<td>30</td>
<td>340</td>
<td>621</td>
</tr>
<tr>
<td>Technip TPG 500 (5)</td>
<td>722</td>
<td>160</td>
<td>680</td>
<td>50</td>
<td>37</td>
<td>0</td>
<td>0</td>
<td>19</td>
<td>340</td>
<td>621</td>
</tr>
<tr>
<td>Technip TPG 500 (6)</td>
<td>722</td>
<td>160</td>
<td>680</td>
<td>30</td>
<td>37</td>
<td>19</td>
<td>30</td>
<td>19</td>
<td>340</td>
<td>621</td>
</tr>
</tbody>
</table>

All dimensions are in millimeters, yield ystresses are in MPa.
Strength interaction equations:

\[
\left\{ \left( \frac{M_x}{M'_{px}} \right)^{1/4} + \left( \frac{M_y}{M'_{py}} \right)^{1/4} \right\} \leq 1.00
\]

where

\[
M'_{py} = M_{py} \left[ 1 - \left( \frac{P}{P_y} \right)^{1.45} \right]
\]

When \( M_x \geq 0 \):

\[
\xi = 1.8 + 2.7 \left( \frac{P}{P_y} \right) + 2.8 \left( \frac{P}{P_y} \right)^2 - 5.6 \left( \frac{P}{P_y} \right)^3
\]

and

\[
M'_{px} = M_{px} \left[ 1 - \left( \frac{P}{P_y} \right)^{1.12} \right]^{1/1.12}
\]

When \( M_x < 0 \):

\[
\xi = 1.8
\]

and for \((P/P_y) \leq 0.25\):

\[
M'_{px} = -M_{px}
\]

for \((P/P_y) > 0.25\):

\[
M'_{px} = -M_{px} \left[ 1 - \left( \frac{P}{0.75P_y} - \frac{1}{3} \right)^{1.45} \right]
\]

Figure F.3-3 — Interaction equations/curves for tubular chords with offset double racks.
Table F.3-3 — Data for tubular chords with offset double racks

Legend: Chord dimensions

All dimensions are in millimeters, Yield Stresses are in MPa

<--- Yield --->

<table>
<thead>
<tr>
<th>Design</th>
<th>D</th>
<th>t1</th>
<th>L1</th>
<th>L2</th>
<th>t2</th>
<th>t3</th>
<th>Fy1</th>
<th>Fy2</th>
<th>Fy3</th>
<th>Bay</th>
<th>Ht</th>
<th>Yena</th>
<th>Yco</th>
<th>S</th>
<th>E</th>
</tr>
</thead>
<tbody>
<tr>
<td>Levingston 011-C</td>
<td>914</td>
<td>29</td>
<td>305</td>
<td>906</td>
<td>127</td>
<td>0</td>
<td>483</td>
<td>621</td>
<td>0</td>
<td>4826</td>
<td>84</td>
<td>100</td>
<td>16</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Levingston 111</td>
<td>1016</td>
<td>32</td>
<td>305</td>
<td>1047</td>
<td>127</td>
<td>0</td>
<td>690</td>
<td>690</td>
<td>0</td>
<td>4877</td>
<td>73</td>
<td>73</td>
<td>0</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Mitsui JC-300 (Key Hawaii)</td>
<td>1016</td>
<td>32</td>
<td>305</td>
<td>1046</td>
<td>127</td>
<td>0</td>
<td>690</td>
<td>690</td>
<td>0</td>
<td>5650</td>
<td>78</td>
<td>78</td>
<td>0</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Mitsui 1-off (Key Bermuda)</td>
<td>1016</td>
<td>29</td>
<td>305</td>
<td>1046</td>
<td>127</td>
<td>0</td>
<td>690</td>
<td>690</td>
<td>0</td>
<td>5050</td>
<td>73</td>
<td>73</td>
<td>0</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Hitachi Drill-Hope</td>
<td>762</td>
<td>30</td>
<td>190</td>
<td>882</td>
<td>127</td>
<td>0</td>
<td>690</td>
<td>690</td>
<td>0</td>
<td>5500</td>
<td>57</td>
<td>57</td>
<td>0</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Hitachi C-150 (Ile Du Levant)</td>
<td>762</td>
<td>30</td>
<td>190</td>
<td>890</td>
<td>130</td>
<td>0</td>
<td>690</td>
<td>690</td>
<td>0</td>
<td>5500</td>
<td>60</td>
<td>60</td>
<td>0</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Hitachi K1040/44/45</td>
<td>900</td>
<td>30</td>
<td>300</td>
<td>882</td>
<td>127</td>
<td>0</td>
<td>690</td>
<td>690</td>
<td>0</td>
<td>5090</td>
<td>77</td>
<td>77</td>
<td>0</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Hitachi K1060 (Sagar Lakshmi)</td>
<td>900</td>
<td>30</td>
<td>300</td>
<td>854</td>
<td>127</td>
<td>13</td>
<td>690</td>
<td>690</td>
<td>690</td>
<td>5260</td>
<td>84</td>
<td>84</td>
<td>0</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Robco 350-C</td>
<td>876</td>
<td>29</td>
<td>292</td>
<td>881</td>
<td>127</td>
<td>0</td>
<td>690</td>
<td>690</td>
<td>0</td>
<td>5461</td>
<td>83</td>
<td>83</td>
<td>0</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Most of leg

Towage Section

Rest of leg

Btm 2 bays

Btm 3 bays

Rest of Leg
Strength interaction equations:

\[
\left( \frac{M_x / M_{px} - K}{M'_{px} / M_{px} - K} \right)^{\xi} + \left( \frac{M_y}{M_{py}} \right)^{1/\xi} \leq 1.00
\]

where:

\[ K = -0.8 \left( \frac{P}{P_y} \right) + 0.4 \left( \frac{P}{P_y} \right)^2 + 0.4 \left( \frac{P}{P_y} \right)^3 \]

and

\[ M'_{py} = M_{py} \left\{ 1 - \left( \frac{P}{P_y} \right)^{2.1} \right\} \]

When \((M_x/M_{px}) \geq K\):

\[ M'_{px} = M_{px} \left\{ 1 - \left( \frac{P}{P_y} \right)^{1.46} \right\} \]

and \(\xi = 1.45\)

When \((M_x/M_{px}) < K\):

\[ M'_{px} = -M_{px} \left\{ 1 - \left( \frac{P}{P_y} \right) \right\}^{1.04} \]

and \(\xi = 1.45 + 2.35 \left( \frac{P}{P_y} \right) + 4.7 \left( \frac{P}{P_y} \right)^2\)

**Figure F.3-4 — Interaction equations/curves for triangular chords with single racks**
### Table F.3-4— Data for triangular chords with single rack

| Design                              | L1  | t1  | L2  | t2  | L3  | t3  | L4  | t4  | L5  | t5  | L6  | t6  | X1  | Y1  | Y2  | Y3  | Fy1 | Fy2 | Fy3 | Fy4 | Fy5 | Bay | Ht  | Yena | Ycos | E   |
|-------------------------------------|-----|-----|-----|-----|-----|-----|-----|-----|-----|-----|-----|-----|-----|-----|-----|-----|-----|-----|-----|-----|-----|-----|-----|-----|-----|
| MarLet Standard (3/4" side plates) | 711 | 51  | 466 | 19  | 213| 127 | 0   | 0   | 0   | 0   | 236| 457 | 0   | 0   | 483 | 453 | 587 | 0   | 0   | 3408| 259| 279 | 20  | 20  | 20  |
| MarLet Standard (7/8" side plates) | 711 | 51  | 466 | 22  | 213| 127 | 0   | 0   | 0   | 0   | 236| 457 | 0   | 0   | 483 | 453 | 587 | 0   | 0   | 3408| 259| 279 | 20  | 20  | 20  |
| MarLet Standard (1" side plates)   | 711 | 51  | 466 | 25  | 213| 127 | 0   | 0   | 0   | 0   | 236| 457 | 0   | 0   | 483 | 453 | 587 | 0   | 0   | 3408| 260| 279 | 19  | 19  | 19  |
| MarLet Standard + side stiffeners  | 711 | 51  | 466 | 19  | 213| 127 | 0   | 127| 25  | 0   | 236| 457 | 211| 483| 453 | 587 | 0   | 483 | 3408| 259| 279 | 19  | 20  | 20  |
| MarLet Std 116 (1"x4" rack stiffeners) | 711 | 51  | 466| 19  | 213| 127 | 102| 25  | 0   | 236| 457 | 524| 483| 453 | 587 | 483 | 0   | 483 | 3408| 260| 279 | 19  | 20  | 20  |
| MarLet 116 North Sea (1"x4"+1"x12" stiffens) | 711 | 51  | 466 | 19  | 213| 127 | 102| 305| 25  | 0   | 236| 457 | 524| 118| 483| 453 | 587 | 483 | 483 | 3408| 276| 291 | 15  | 20  | 20  |
| MarLet 116 (1"x4"+1.5"x12" stiffens) | 711 | 51  | 466 | 19  | 213| 127 | 102| 305| 38  | 0   | 236| 457 | 524| 124| 483| 453 | 587 | 483 | 483 | 3408| 290| 304 | 14  | 20  | 20  |
| MarLet Gorilla (150-88)            | 813 | 76  | 573 | 57  | 222| 140 | 0   | 0   | 0   | 0   | 248| 600 | 0   | 0   | 483 | 483 | 620 | 0   | 0   | 5113| 302| 323 | 21  | 20  | 20  |
| MarLet Super 300                    | 813 | 76  | 607 | 38  | 222| 140 | 0   | 305| 51  | 0   | 268| 600 | 296| 483| 483 | 620 | 0   | 483 | 5113| 298| 323 | 25  | 20  | 20  |
| LeTourneau 150                     | 711 | 64  | 441 | 38  | 213| 127 | 0   | 0   | 0   | 0   | 218| 457 | 0   | 0   | 414 | 414 | 620 | 0   | 0   | 2556| 245| 245 | 0   | 0   | 0   |
| LeTourneau 150 (3/4" side pl)      | 711 | 51  | 466 | 19  | 213| 127 | 0   | 0   | 0   | 0   | 236| 457 | 0   | 0   | 414 | 414 | 620 | 0   | 0   | 2556| 259| 303 | 44  | 20  | 20  |
| LeTourneau 150 (1.125" side pl)    | 711 | 51  | 466 | 29  | 213| 127 | 0   | 0   | 0   | 0   | 236| 457 | 0   | 0   | 414 | 414 | 620 | 0   | 0   | 2556| 259| 303 | 44  | 20  | 20  |
| LeTourneau 46,47                   | 559 | 44  | 432 | 13  | 178| 89  | 0   | 0   | 0   | 0   | 166| 432 | 0   | 0   | 872 | 872 | 620 | 0   | 0   | 3456| 315| 315 | 0   | 0   | 0   |
| Mitsubishi MD-T76J                 | 50  | 574 | 25  | 225| 125 | 0   | 0   | 0   | 0   | 226| 575 | 0   | 0   | 687 | 687 | 687 | 0   | 0   | 3456| 315| 315 | 0   | 0   | 0   |
| Gusto 1-off: (Maersk Endeavour)    | 800 | 60  | 592 | 30  | 283| 127 | 0   | 0   | 0   | 0   | 359| 443 | 0   | 0   | 620 | 620 | 620 | 0   | 0   | 4800| 284| 284 | 0   | 0   | 0   |
| Gusto 1-off: (Maersk Explorer)     | 800 | 90  | 534 | 40  | 283| 127 | 0   | 0   | 0   | 0   | 331| 443 | 0   | 0   | 620 | 620 | 620 | 0   | 0   | 4800| 255| 255 | 0   | 0   | 0   |
| Gusto 1-off: (Maersk Endavour)     | 800 | 110 | 488 | 50  | 283| 127 | 0   | 0   | 0   | 0   | 307| 443 | 0   | 0   | 620 | 620 | 620 | 0   | 0   | 4800| 244| 244 | 0   | 0   | 0   |
| Gusto 1-off: (Maersk Explorer)     | 800 | 78  | 535 | 38  | 279| 127 | 0   | 0   | 0   | 0   | 337| 453 | 0   | 0   | 690 | 690 | 690 | 0   | 0   | 4539| 268| 268 | 0   | 0   | 0   |
| BMC 1-off design (Trident 7)       | 711 | 38  | 356 | 19  | 279| 127 | 0   | 0   | 0   | 0   | 256| 204 | 0   | 0   | 3353| 197| 197 | 0   | 0   | 0   | 0   | 0   |

All dimensions are in millimeters, Yield Stresses are in MPa
Annex G
(informative)

Contents list for typical site assessment report
G.1 General

This annex provide an outline for the contents of a site-specific assessment report.

### G.2 Jack-up and location:

<table>
<thead>
<tr>
<th>Rig Name:</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Rig Type:</td>
<td></td>
</tr>
<tr>
<td>Operator:</td>
<td></td>
</tr>
<tr>
<td>Location Name:</td>
<td></td>
</tr>
<tr>
<td>Latitude</td>
<td></td>
</tr>
<tr>
<td>Longitude</td>
<td></td>
</tr>
</tbody>
</table>

### G.3 Data check:

<table>
<thead>
<tr>
<th>Jack-up data:</th>
<th>Exists?</th>
</tr>
</thead>
<tbody>
<tr>
<td>Is Jack-up in Class?</td>
<td>y</td>
</tr>
<tr>
<td>Soils data</td>
<td>X</td>
</tr>
<tr>
<td>Prior experience</td>
<td>x</td>
</tr>
<tr>
<td>Adjacent infrastructure</td>
<td></td>
</tr>
<tr>
<td>Metocean data</td>
<td>x</td>
</tr>
<tr>
<td>Arrangement at location:</td>
<td></td>
</tr>
<tr>
<td>Rig heading:</td>
<td></td>
</tr>
<tr>
<td>Water depth:</td>
<td></td>
</tr>
<tr>
<td>Hull clearance above LAT</td>
<td></td>
</tr>
<tr>
<td>Conductor top tension/ support mechanism</td>
<td></td>
</tr>
<tr>
<td>Conductor diameter and number</td>
<td></td>
</tr>
<tr>
<td>Earthquake data</td>
<td></td>
</tr>
<tr>
<td>Accidental situations</td>
<td></td>
</tr>
<tr>
<td>Operator Requirements (e.g. required airgap)</td>
<td></td>
</tr>
<tr>
<td>Agreed consequence class:</td>
<td></td>
</tr>
</tbody>
</table>

### G.4 Site assessment results summary

<table>
<thead>
<tr>
<th>Site assessment results summary</th>
<th>Acceptable?</th>
</tr>
</thead>
<tbody>
<tr>
<td>Is jack-up suitable for site and time of year?</td>
<td></td>
</tr>
<tr>
<td>Are there specific seasonal and/or operational restrictions or limitations</td>
<td></td>
</tr>
<tr>
<td>Minimum leg reserve above upper guide</td>
<td></td>
</tr>
<tr>
<td>--------------------------------------</td>
<td>--</td>
</tr>
<tr>
<td>Foundation fixity used?</td>
<td>Y</td>
</tr>
<tr>
<td>Preload OK</td>
<td>No</td>
</tr>
<tr>
<td>Foundation OK</td>
<td>yes</td>
</tr>
<tr>
<td>Member strength OK</td>
<td>x</td>
</tr>
<tr>
<td>Overturning OK</td>
<td>x</td>
</tr>
<tr>
<td>Possible infrastructure interaction</td>
<td></td>
</tr>
<tr>
<td>Hull displacement</td>
<td></td>
</tr>
<tr>
<td>Earthquake</td>
<td></td>
</tr>
<tr>
<td>Accidental</td>
<td></td>
</tr>
</tbody>
</table>

**G.5 Jack-up data:**

<table>
<thead>
<tr>
<th>G.6</th>
</tr>
</thead>
<tbody>
<tr>
<td>Length</td>
</tr>
<tr>
<td>Breadth</td>
</tr>
<tr>
<td>Depth</td>
</tr>
<tr>
<td>Installed leg length</td>
</tr>
<tr>
<td>No of Legs</td>
</tr>
<tr>
<td>No of Chords/leg (1-4)</td>
</tr>
<tr>
<td>Longitudinal Leg Spacing</td>
</tr>
<tr>
<td>Transverse Leg Spacing</td>
</tr>
<tr>
<td>Chord Spacing</td>
</tr>
<tr>
<td>Reference point for chord spacing, e.g. pitch points</td>
</tr>
<tr>
<td>Weight of one leg including spudcan, including permanent ballast, but excluding water ballast and buoyancy</td>
</tr>
<tr>
<td>Weight of one spudcan, including permanent ballast, but excluding water ballast and buoyancy</td>
</tr>
<tr>
<td>Are legs (not spudcans) free-flooding</td>
</tr>
<tr>
<td>Type of holding system (jacks or chocks)</td>
</tr>
<tr>
<td>Number of jacks per leg</td>
</tr>
<tr>
<td>Jack holding strength (jacking)</td>
</tr>
<tr>
<td>Jack holding strength (Design maximum holding)</td>
</tr>
<tr>
<td>Jack holding strength (preload holding)</td>
</tr>
<tr>
<td>Jack holding strength (ultimate)</td>
</tr>
<tr>
<td>Light ship</td>
</tr>
<tr>
<td></td>
</tr>
<tr>
<td>--------------------------</td>
</tr>
<tr>
<td><strong>Movable fixed load</strong></td>
</tr>
<tr>
<td><strong>Variable load</strong></td>
</tr>
<tr>
<td><strong>Total maximum hull weight</strong></td>
</tr>
<tr>
<td><strong>Total minimum hull weight</strong></td>
</tr>
<tr>
<td><strong>Overall hull centre of gravity (and tolerance where applicable)</strong></td>
</tr>
<tr>
<td><strong>Total available preload</strong></td>
</tr>
<tr>
<td><strong>Type of preload procedure (e.g. one leg at a time)</strong></td>
</tr>
<tr>
<td><strong>Maximum preload spudcan reactions at the sea bed using chosen preload method (including leg/spudcan weight and buoyancy)</strong></td>
</tr>
<tr>
<td></td>
</tr>
<tr>
<td></td>
</tr>
<tr>
<td></td>
</tr>
<tr>
<td></td>
</tr>
<tr>
<td></td>
</tr>
<tr>
<td><strong>Spudcan Diameter</strong></td>
</tr>
<tr>
<td><strong>Spudcan Height</strong></td>
</tr>
<tr>
<td><strong>Spudcan Volume</strong></td>
</tr>
<tr>
<td><strong>Maximum bearing area of spudcan</strong></td>
</tr>
<tr>
<td><strong>Distance from spudcan maximum bearing area to tip</strong></td>
</tr>
<tr>
<td><strong>Advertised Operating Water depth</strong></td>
</tr>
<tr>
<td><strong>Designer</strong></td>
</tr>
<tr>
<td><strong>Class/type</strong></td>
</tr>
<tr>
<td><strong>Classification Society</strong></td>
</tr>
</tbody>
</table>

**G.7 Arrangements at location**

<p>| | |</p>
<table>
<thead>
<tr>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Installed Leg Length</strong></td>
<td></td>
</tr>
<tr>
<td><strong>Distance from keel to top of upper guide</strong></td>
<td></td>
</tr>
<tr>
<td><strong>Hull Clearance above LAT</strong></td>
<td></td>
</tr>
<tr>
<td><strong>Water Depth</strong></td>
<td></td>
</tr>
<tr>
<td><strong>Expected Penetration with full Preload</strong></td>
<td></td>
</tr>
<tr>
<td><strong>Reserve of Leg</strong></td>
<td></td>
</tr>
</tbody>
</table>

Expanded the following table as necessary to account for directional and seasonal data
## G.9 Metocean conditions

<table>
<thead>
<tr>
<th>Question</th>
<th>Answer</th>
</tr>
</thead>
<tbody>
<tr>
<td>50 year independent extremes or 100 year Joint Probability?</td>
<td></td>
</tr>
<tr>
<td>Load factor</td>
<td></td>
</tr>
<tr>
<td>Has directional metocean data been used?</td>
<td></td>
</tr>
<tr>
<td>Has seasonal metocean data been used?</td>
<td></td>
</tr>
<tr>
<td>Water depth</td>
<td></td>
</tr>
<tr>
<td>Wave Details</td>
<td></td>
</tr>
<tr>
<td>Maximum Wave Height</td>
<td></td>
</tr>
<tr>
<td>Associated Wave Period (intrinsic)</td>
<td></td>
</tr>
<tr>
<td>Significant Wave Height</td>
<td></td>
</tr>
<tr>
<td>Peak Period (intrinsic)</td>
<td></td>
</tr>
<tr>
<td>Wave Crest Height</td>
<td></td>
</tr>
<tr>
<td>Wind Speed (at 10 metres above water level, collinear with wave)</td>
<td></td>
</tr>
<tr>
<td>1 Hour wind speed</td>
<td></td>
</tr>
<tr>
<td>1 Minute wind speed (required)</td>
<td></td>
</tr>
<tr>
<td>3 second gust</td>
<td></td>
</tr>
<tr>
<td>Surge</td>
<td></td>
</tr>
<tr>
<td>Tide</td>
<td></td>
</tr>
<tr>
<td>Reserve on Hull Clearance</td>
<td></td>
</tr>
<tr>
<td>Hull Elevation above LAT</td>
<td></td>
</tr>
<tr>
<td>Expected storm settlement</td>
<td></td>
</tr>
<tr>
<td>Other allowances e.g. reservoir settlement</td>
<td></td>
</tr>
<tr>
<td>Surface Current (collinear with wind and wave)</td>
<td></td>
</tr>
<tr>
<td>Bottom Current (collinear with wind and wave)</td>
<td></td>
</tr>
<tr>
<td>Current profile details</td>
<td></td>
</tr>
<tr>
<td>Marine Growth</td>
<td></td>
</tr>
<tr>
<td>Profile</td>
<td></td>
</tr>
<tr>
<td>Predeployment Marine Growth Profile</td>
<td></td>
</tr>
<tr>
<td>Are there operational restrictions (e.g. variable load limits, heading, air gap, leg/guide location, etc.)?</td>
<td></td>
</tr>
<tr>
<td>Are there specific Operator requirements that may affect the suitability for the location?</td>
<td></td>
</tr>
</tbody>
</table>

## G.11 Site investigation

<table>
<thead>
<tr>
<th>Activity</th>
<th>Exist?</th>
<th>Year</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bathymetry survey</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
### G.12 Site hazards

- Pipeline hazards
- Adjacent structures
- Location move-on hazards

### G.13 Soils

- Distance from location of survey
- General soils description
- Location move-on hazards
- Profile details
- Variability over area
- Confidence in data
- Previous experience at the location
- Load penetration curve
- Range of predicted penetrations after preloading
- Is there a punchthrough potential during installation
  - Method for mitigating hazards
- Is there a risk of punchthrough or significant settlement should the foundation reactions exceed capacity developed by preloading.
  - Method for mitigating hazards
- Does the predicted penetration curve show potential for punchthrough or significant settlement (precipitous settlement) should the foundation reactions exceed those assessed (for information only; there is no acceptance criterion).
  - Method for mitigating hazards
- Previous spudcan holes
  - Method for mitigating hazards
- Other geotechnical hazards?
  - Method for mitigating hazards
- Is there scour potential
  - Method for mitigating hazards
NOTE << The potential for punchthrough should also be included in the results section, and may need to be added to the main text >>

<table>
<thead>
<tr>
<th><strong>G.14 Analysis path/route/assumptions</strong></th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>G.15</strong></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th><strong>G.16 Spudcan fixity used in analysis</strong></th>
<th><strong>G.17</strong></th>
</tr>
</thead>
<tbody>
<tr>
<td>Initial Stiffness (for each spudcan if soils differ)</td>
<td></td>
</tr>
<tr>
<td>Rotational</td>
<td></td>
</tr>
<tr>
<td>Lateral</td>
<td></td>
</tr>
<tr>
<td>Vertical</td>
<td></td>
</tr>
<tr>
<td>Ultimate Capacity</td>
<td></td>
</tr>
<tr>
<td>Rotational</td>
<td></td>
</tr>
<tr>
<td>Lateral</td>
<td></td>
</tr>
<tr>
<td>Vertical</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th><strong>G.18 Earthquake analysis</strong></th>
</tr>
</thead>
<tbody>
<tr>
<td>Does location fall below the 19905-1 cut-off (19901-2 seismicity level ??2??) ?</td>
</tr>
<tr>
<td>Was an earthquake analysis performed?</td>
</tr>
<tr>
<td>Source of the earthquake data</td>
</tr>
<tr>
<td>What vertical ground motions were used (in many cases the critical condition)?</td>
</tr>
<tr>
<td>Was vertical spectrum some ratio of lateral ground motion spectrum, and if so, how was it derived?</td>
</tr>
<tr>
<td>Is spudcan fixity different from Metocean analysis, and if so what value was used?</td>
</tr>
<tr>
<td>Was linear analysis sufficient to prove acceptability of location?</td>
</tr>
<tr>
<td>Describe nonlinear analysis, if used</td>
</tr>
</tbody>
</table>
### G.19 Accidental situations

<table>
<thead>
<tr>
<th>Question</th>
<th>Answer</th>
</tr>
</thead>
<tbody>
<tr>
<td>Were any accidental situations assessed (e.g. collision)?</td>
<td></td>
</tr>
<tr>
<td>What were the results of the analyses?</td>
<td></td>
</tr>
</tbody>
</table>

### G.20 Intermediate results

<table>
<thead>
<tr>
<th>Topic</th>
<th>Details</th>
</tr>
</thead>
<tbody>
<tr>
<td>Natural period (with P-Δ)</td>
<td></td>
</tr>
<tr>
<td>DAF If SDOF on BS</td>
<td></td>
</tr>
<tr>
<td>Stability of DAF with simulation duration (2-stage)</td>
<td></td>
</tr>
<tr>
<td>Stability of static &amp; dynamic MPME (1-stage)</td>
<td></td>
</tr>
<tr>
<td>If Random on BS &amp; OTM</td>
<td></td>
</tr>
<tr>
<td>Factored Wind, Wave/current, Inertial BS's &amp; OTM's</td>
<td></td>
</tr>
</tbody>
</table>

### G.21 Analysis results (utilisation checks)

<table>
<thead>
<tr>
<th>Topic</th>
<th>Details</th>
</tr>
</thead>
<tbody>
<tr>
<td>Preload</td>
<td></td>
</tr>
<tr>
<td>Foundation:</td>
<td></td>
</tr>
<tr>
<td>Bearing check</td>
<td></td>
</tr>
<tr>
<td>Sliding check</td>
<td></td>
</tr>
<tr>
<td>Magnitude of additional penetration under storm loads</td>
<td></td>
</tr>
<tr>
<td>Overturning</td>
<td></td>
</tr>
<tr>
<td>Chord Member strength</td>
<td></td>
</tr>
<tr>
<td>Horizontal member strength</td>
<td></td>
</tr>
<tr>
<td>Diagonal member strength</td>
<td></td>
</tr>
<tr>
<td>Jacks/rack chocks</td>
<td></td>
</tr>
<tr>
<td>Results of Earthquake Analysis (if applicable)</td>
<td></td>
</tr>
</tbody>
</table>
Bibliography

<< Note: Some bibliographic references to be completed / checked >>


NOTE << or should this be: Svano G. (1996), "Foundation Fixity Study for Jack-up Unit", SINTEF report STF22 F96660, August 1996.>>


NOTE << Are all the six references that follow supposed to be referenced from A.9.3? Presently they are not, but they were in the reference list provided for that clause by the Technical Panel >>


[A.9.4-25] GEO/DGI << details required >>.


[A.9.4-27] OGP << details required >>.


NOTE << Reference was provided by Jack Templeton, noting that "At the moment, my attribution of the equation given to this particular paper is indirect, but I believe that it is the correct original reference, and I will try to retrieve the actual paper to make sure it is correct." >>


[D.1-2] << to be provided >>

[D.1-3] << to be provided >>


