
**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





COPYRIGHT PROTECTED DOCUMENT

© ISO 2012

All rights reserved. Unless otherwise specified, no part of this publication may be reproduced or utilized in any form or by any means, electronic or mechanical, including photocopying and microfilm, without permission in writing from either ISO at the address below or ISO's member body in the country of the requester.

ISO copyright office
Case postale 56 • CH-1211 Geneva 20
Tel. + 41 22 749 01 11
Fax + 41 22 749 09 47
E-mail copyright@iso.org
Web www.iso.org

Published in Switzerland

Contents

Page

Foreword	v
Introduction.....	vii
1 Scope	1
2 Normative references	2
3 Terms and definitions	2
4 Symbols and abbreviated terms	13
4.1 Symbols.....	13
4.2 Abbreviated terms	14
5 Overall considerations	15
5.1 General	15
5.2 Assessment approach	16
5.3 Selection of limit states	18
5.4 Determination of assessment situations	18
5.5 Exposure levels	20
5.6 Analytical tools	23
6 Data to assemble for each site.....	24
6.1 Applicability	24
6.2 Jack-up data.....	24
6.3 Site and operational data.....	24
6.4 Metocean data.....	24
6.5 Geophysical and geotechnical data	25
6.6 Earthquake data.....	26
7 Actions.....	26
7.1 Applicability	26
7.2 General	26
7.3 Metocean actions	27
7.4 Functional actions.....	28
7.5 Displacement dependent effects	28
7.6 Dynamic effects	28
7.7 Earthquakes	28
7.8 Other actions	28
8 Structural modelling.....	28
8.1 Applicability	28
8.2 Overall considerations	28
8.3 Modelling the leg	29
8.4 Modelling the hull	30
8.5 Modelling the leg-to-hull connection.....	30
8.6 Modelling the spudcan and foundation	31
8.7 Mass modelling.....	32
8.8 Application of actions	32
9 Foundations	35
9.1 Applicability	35
9.2 General	35
9.3 Geotechnical analysis of independent leg foundations	36
9.4 Other considerations	39
10 Structural response.....	41
10.1 Applicability	41

10.2	General considerations	41
10.3	Types of analyses and associated methods.....	41
10.4	Common parameters	42
10.5	Storm analysis	44
10.6	Fatigue analysis	47
10.7	Earthquake analysis	47
10.8	Accidental situations.....	47
10.9	Alternative analysis methods	48
11	Long-term applications	48
11.1	Applicability.....	48
11.2	Assessment data	48
11.3	Special requirements	49
11.4	Survey requirements	50
12	Structural strength.....	50
12.1	Applicability.....	50
12.2	Classification of member cross-sections	51
12.3	Section properties of non-circular prismatic members.....	52
12.4	Effects of axial force on bending moment	53
12.5	Strength of tubular members	53
12.6	Strength of non-circular prismatic members.....	53
12.7	Assessment of joints.....	53
13	Acceptance criteria.....	54
13.1	Applicability.....	54
13.2	General formulation of the assessment check.....	55
13.3	Leg strength assessment	55
13.4	Spudcan strength assessment.....	56
13.5	Holding system strength assessment.....	56
13.6	Hull elevation assessment.....	56
13.7	Leg length reserve assessment	56
13.8	Overturning stability assessment.....	57
13.9	Foundation integrity assessment	57
13.10	Interaction with adjacent infrastructure	58
13.11	Temperatures	59
Annex A (informative) Additional information and guidance		60
Annex B (normative) Summary of partial action and partial resistance factors		238
Annex C (informative) Additional information on structural modelling and response analysis.....		240
Annex D (informative) Foundations — Recommendations for the acquisition of site-specific geotechnical data.....		250
Annex E (informative) Foundations — Additional information and alternative approaches.....		256
Annex F (informative) Informative annex on Clause A.12 — Structural strength		269
Annex G (informative) Contents list for typical site-specific assessment report.....		283
Annex H (informative) Regional information		290
Bibliography.....		299

Foreword

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

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

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

Attention is drawn to the possibility that some of the elements of this document may be the subject of patent rights. ISO shall not be held responsible for identifying any or all such patent rights.

ISO 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 consists of the following parts, under the general title *Petroleum and natural gas industries — Site-specific assessment of mobile offshore units*:

- *Part 1: Jack-ups*
- *Part 2: Jack-ups commentary and detailed sample calculation* [Technical Report]

The following part is under preparation:

- Part 3, dealing with the site-specific assessment of mobile floating units.

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

- ISO 19900, *Petroleum and natural gas industries — General requirements for offshore structures*
- ISO 19901-1, *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 19905-1:2012(E)

- 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 19901-8¹⁾, *Petroleum and natural gas industries — Specific requirements for offshore structures — Part 8: Marine soils investigations*
- ISO 19902, *Petroleum and natural gas industries — Fixed steel offshore structures*
- ISO 19903, *Petroleum and natural gas industries — Fixed concrete offshore structures*
- ISO 19904-1, *Petroleum and natural gas industries — Floating offshore structures — Part 1: Monohulls, semi-submersibles and spars*
- ISO 19905-1, *Petroleum and natural gas industries — Site-specific assessment of mobile offshore units — Part 1: Jack-ups*
- ISO/TR 19905-2, *Petroleum and natural gas industries — Site-specific assessment of mobile offshore units — Part 2: Jack-ups commentary and detailed sample calculation*
- ISO 19905-3²⁾, *Petroleum and natural gas industries — Site-specific assessment of mobile offshore units — Part 3: Floating units*
- ISO 19906, *Petroleum and natural gas industries — Arctic offshore structures*

1) Under preparation. It is also expected that there will be further parts of ISO 19901.

2) Under preparation.

Introduction

The series of International Standards applicable to types of offshore structure, ISO 19900 to ISO 19906, addresses design requirements and assessments for all offshore structures used by the petroleum and natural gas industries worldwide. Through their application, the intention is to achieve reliability levels appropriate for 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 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 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 of ISO 19905, which has been developed from SNAME Technical & Research Bulletin 5-5A^[7], 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.

NOTE For the exposure level 1(L1) assessment and, where appropriate, the exposure level 2 (L2) assessment prior to evacuation being effected, this part of ISO 19905 requires the use of 50 year independent or 100 year joint probability metocean extremes, together with associated partial action factors. It is based on extensive benchmarking and best practice in the international community.

Site-specific assessment is normally carried out when 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, different practices or standards could have been applied.

The purpose of the site assessment is to demonstrate the adequacy of the jack-up and its foundations for the assessment situations and defined limit states, taking into account the consequences of failure. It is important that the results of a site-specific assessment be appropriately recorded and communicated to those persons required to know or act on the conclusions and recommendations. Alternative approaches to the site-specific assessment can be used, provided that they have been shown to give a level of structural reliability equivalent, or superior, to that implicit in this part of ISO 19905.

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

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

To meet certain needs of industry for linking software to specific elements in this part of ISO 19905, a special numbering system has been permitted for figures, tables, equations and bibliographic references.

In International Standards, the following verbal forms are used:

- “shall” and “shall not” are used to indicate requirements strictly to be followed in order to conform to the document and from which no deviation is permitted;

- “should” and “should not” are used to indicate that, among several possibilities, one is recommended as particularly suitable, without mentioning or excluding others, or that a certain course of action is preferred but not necessarily required, or that (in the negative form) a certain possibility or course of action is deprecated but not prohibited;
- “may” is used to indicate a course of action permissible within the limits of the document;
- “can” and “cannot” are used for statements of possibility and capability, whether material, physical or causal.

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

Part 1: Jack-ups

1 Scope

This part of ISO 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 part of ISO 19905 form an integrated approach, which is used in its entirety for the site-specific assessment of a jack-up.

This part of ISO 19905 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, it is intended that the assessor supplement the provisions of this part of ISO 19905 with the provisions relating to ice actions and procedures for ice management contained in ISO 19906.

This part of ISO 19905 does not address design, transportation to and from site, or installation and removal from site. However, it is advisable 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 of ISO 19905 is applicable only to independent leg jack-ups that either

- hold a valid classification society certification from a recognized classification society (RCS) throughout the duration of the operation at the specific site subject to assessment; or
- have been verified by an independent competent body to be structurally fit for purpose for elevated situations and are subject to periodic inspection, both to the standards of an RCS.

NOTE 1 An RCS is an International Association of Classification Societies (IACS) member body, meeting the RCS definition given in 3.52.

Jack-ups that do not comply with this requirement are assessed according to the provisions of ISO 19902, supplemented by methodologies from this part of ISO 19905, where applicable.

NOTE 2 Future revisions of this part of ISO 19905 can 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 part of ISO 19905. Annex A provides references to other publications addressing this topic.

NOTE 4 RCS rules and the IMO MODU code provide guidance for the design of jack-ups.

2 Normative references

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

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

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

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

ISO 19902, *Petroleum and natural gas industries — Fixed steel 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 19902, and the following apply.³⁾

3.1

abnormal wave crest

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

3.2

accidental situation

exceptional situation of the structure

EXAMPLE Impact; fire; explosion; local failure; 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, definition 2.1]

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 part of ISO 19905

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

3.5

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 part of ISO 19905 to demonstrate that the relevant limit states are not exceeded.

3) Other terms and definitions relevant for the use of this part of ISO 19905 are also found in ISO 19901-4 and ISO 19906.

3.6**assessor**

entity performing the site-specific assessment

3.7**backfill**

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

NOTE Backfilling can occur during or after preloading. $W_{BF,0}$ refers to the submerged weight of the backfilling that occurs up to achieving the preload reaction. $W_{BF,A}$ refers to the submerged weight of the backfilling that occurs after the maximum preload has been applied and held. Both $W_{BF,0}$ and $W_{BF,A}$ can comprise backflow and/or infill. For discussion of the effects, see A.9.3.2.1.4.

3.8**backflow**

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

NOTE Backflow is part of **backfill** (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, definition 2.5]

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 can be used to generate reaction forces at locations of restraint.

[ISO 19902:2007, definition 3.6]

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, definition 3.2]

3.12**consequence category**

classification system for identifying the environmental, economic and indirect personnel safety consequences of failure of a jack-up

NOTE 1 Categories for environmental and economic consequences are the following (see 5.3.3):

- C1: high environmental or economic consequence;
- C2: medium environmental or economic consequence;
- C3: low environmental or economic consequence.

NOTE 2 Adapted from ISO 19902:2007, definition 3.11.

3.13

critical component

structural component, failure of which could cause failure of the whole structure, or a significant part of it

NOTE A critical component is part of the primary structure.

[ISO 19902:2007, definition 3.12]

3.14

dynamic amplification factor

DAF

ratio of a dynamic action effect to the corresponding static action effect

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

NOTE 2 The DAF excluding the mean values, $K_{DAF,SDOF}$, can typically be obtained from a single degree-of-freedom (SDOF) calculation. In this case, it is defined as the ratio of the amplitude of a dynamic action effect to the amplitude of the corresponding static action effect for periodic excitation of a linear one degree-of-freedom model approximation of jack-up behaviour.

NOTE 3 The DAF including the mean values, $K_{DAF,RANDOM}$, can typically be obtained from a random wave calculation. In this case, it is defined as the ratio of the absolute value of a dynamic action effect to the absolute value of the corresponding static action effect, each including their mean value.

NOTE 4 Adapted from ISO 19902:2007, definition 3.16.

3.15

deterministic analysis

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

3.16

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

NOTE 1 An exposure level 1 (L1) jack-up is the most critical and exposure level 3 (L3) the least (see 5.5).

NOTE 2 Adapted from ISO 19902:2007, definition 3.18.

3.17

extreme storm event

extreme combination of wind, wave and current conditions to which the structure can be subjected during its deployment

NOTE This is the metocean event used for ULS storm assessment (see 6.4).

3.18

fixed load

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

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

3.19

footprint

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

3.20

foundation

soil and spudcan supporting a jack-up leg

3.21**foundation fixity**

rotational restraint offered by the soil to the spudcan

3.22**foundation stability**

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

3.23**global analysis**

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

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

NOTE 2 Adapted from ISO 19902:2007, definition 3.23.

3.24**independent leg jack-up**

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

3.25**inertial loadset**

set of actions that approximates the effect of the inertial forces

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

3.26**infill**

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

NOTE Infill is part of **backfill** (3.7).

3.27**intrinsic wave frequency**

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

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

3.28**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.

3.29**jack-up owner**

representative of the companies owning or chartering the jack-up

3.30**joint probability metocean data**

combinations of wind, wave and current that produce the action effect that can be expected to occur at a site, on average, once in the return period

3.31

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

3.32

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 the following (see 5.5.2):

- S1: manned non-evacuated;
- S2: manned evacuated;
- S3: unmanned.

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

3.33

limit state

state beyond which the structure no longer fulfils the relevant assessment criteria

NOTE Adapted from ISO 19900:2002, definition 2.21.

3.34

load case

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

[ISO 19902:2007, definition 3.29]

3.35

long-term operation

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

3.36

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, definition 3.12]

3.37

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

3.38

mean high water spring tidal level

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

3.39

mean low water spring tidal level

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

3.40 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, definition 3.15]

3.41 mean zero-upcrossing period

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

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

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

where

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

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

m_2 is the second spectral moment;

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

ω is the wave frequency in radians per second.

NOTE 2 Adapted from ISO 19901-1:2005, definition 3.17.

3.42 most probable maximum extreme MPME

value of the maximum of a variable with the highest probability of occurring over a defined period of time

NOTE 1 A defined period of time can be, for example, X hours.

NOTE 2 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 3 Adapted from ISO 19901-1:2005, definition 3.19.

3.43 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.

3.44 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, definition 3.34]

3.45
operating manual
marine operations manual

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 are provided.

3.46
operator
representative of the companies leasing the site

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

3.47
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 through to the time when the maximum load is applied and held

NOTE Whilst three-legged jack-ups preload by taking water ballast on board, jack-ups with four or more legs typically achieve foundation preload by carrying the hull weight on pairs of legs in turn. This procedure is known as pre-driving and generally does not require the addition of water ballast. For the purposes of this part of ISO 19905, no distinction is made between preload and pre-drive.

3.48
preload reaction

maximum vertical reaction under a spudcan, V_{Lo} , supporting 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, reduced by the buoyancy in water of the legs and spudcans (calculated from their external dimensions). Soil buoyancy and the weight of any soil backfill above the spudcan are neglected. It is necessary to take care when accounting for water contained in the spudcan (in some cases this can be included in the quoted leg weight).

NOTE 2 This is the maximum reaction on a spudcan, V_{Lo} , that would be obtained during preloading if the jack-up were installed on an infinitely rigid foundation.

3.49
punch-through

rapid, uncontrolled vertical leg movement due to soil failure in strong soil overlying weak soil

3.50
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.

3.51
rack phase difference
RPD

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

NOTE This is the out-of-plane distortion of the plan-frame.

3.52**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.

3.53**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, definition 3.39]

3.54**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 part of ISO 19905 includes such agencies.

NOTE 3 Within this part of ISO 19905, 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.

NOTE 4 Adapted from ISO 19902:2007, definition 3.40.

3.55**representative value**

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

[ISO 19900:2002, definition 2.26]

3.56**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 exceedance of the event.

[ISO 19901-1:2005, definition 3.23]

3.57**scatter diagram**

joint probability of two or more (metocean) parameters

NOTE 1 A scatter diagram is especially used with wave parameters in the metocean context, see ISO 19901-1:2005, A.5.8. The wave scatter diagram is commonly understood to be the probability of the joint occurrence of the significant wave height (H_s) and a representative period ($T_{z,i}$ or $T_{p,i}$).

NOTE 2 Adapted from ISO 19901-1:2005, definition 3.24.

3.58

scour

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

3.59

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, definition 3.26]

3.60

shallow gas

gas pockets or entrapped gas below impermeable layers at shallow depth

3.61

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 19901-1:2005, definition 3.31) or 4σ , where σ is the standard deviation of the time series of water surface elevation over the duration of the measurement, typically a period of approximately 30 min.

[ISO 19901-1:2005, definition 3.30]

3.62

skirted spudcan

spudcan with a peripheral skirt

3.63

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° . 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.

3.64

sliding

horizontal movement of a spudcan

3.65

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.

3.66

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

[ISO 19901-1:2005, definition 3.33]

3.67

spectral peak period

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

NOTE 1 In practice, there is often more than one peak in a spectrum.

NOTE 2 There are two types of spectral peak period used within this part of ISO 19905: intrinsic and apparent. The distinction is discussed in A.7.3.3.5, which is, in turn, based on ISO 19901-1:2005, 8.3 and A.8.3.

NOTE 3 Adapted from ISO 19901-1:2005, definition 3.32.

3.68

spudcan

structure at the base of a leg supported by the soil

3.69

squeezing

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

3.70

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 non-linear stochastic analysis can only use time domain simulations. This part of ISO 19905 does not support frequency domain stochastic analysis.

3.71

stress concentration factor

SCF

factor relating a nominal stress to the local stress at a detail

NOTE Adapted from ISO 19902:2007, definition 3.50.

3.72

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.

3.73

structural component

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 can be of a different yield strength. See also further discussion in A.12.1.1.

3.74

**structural member
member**

physically distinguishable part of a braced structure connecting two joints

NOTE 1 A structural member can also be defined as the leg of a non-truss leg jack-up.

NOTE 2 See also further discussion in A.12.1.1.

3.75

**sudden hurricane
sudden cyclone
sudden typhoon**

sudden tropical revolving storm that forms near the site and that 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.

3.76

sustained wind speed

time-averaged wind speed with a defined averaging duration of 1 min or longer

NOTE Adapted from ISO 19901-1:2005, definition 3.37, which references a duration of “10 min or longer”.

3.77

undrained shear strength

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

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

[ISO 19901-4:2003, definition 3.9]

3.78

**utilization
member utilization
foundation 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 $U \leq 1,0$, then the utilization is equal to U .

NOTE 5 Adapted from ISO 19902:2007, definition 3.56.

3.79

variable load

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

3.80

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.

4 Symbols and abbreviated terms

4.1 Symbols

B_S	soil buoyancy of spudcan below bearing area, i.e. the submerged weight of soil displaced by the spudcan below D , the greatest depth of maximum cross-sectional spudcan bearing area below the sea floor
C_m	moment reduction factor
D	greatest depth of maximum cross-sectional spudcan bearing area below the sea floor
D_e	equivalent set of inertial actions representing dynamic extreme storm effects or ground motion effects due to earthquakes
E_e	metocean actions due to the extreme storm event
f_{FD}	fatigue damage design factor
F_d	assessment load case
F_H	horizontal force applied to the spudcan due to the assessment load case (see 8.8)
F_V	gross vertical force acting on the soil beneath the spudcan due to the assessment load case F_d (see 8.8)
G_F	actions due to the fixed load positioned such as to adequately represent their vertical and horizontal distribution
G_V	actions due to maximum or minimum variable load, as appropriate, positioned at the most onerous centre of gravity location applicable to the configurations under consideration
$K_{DAF,RANDOM}$	DAF from random wave time domain (stochastic) analyses
$K_{DAF,SDOF}$	DAF from single degree-of-freedom representation of dynamic behaviour
L_{AE}	length of the vector from a specified origin to the action effect
L_{IS}	length of the vector from the same origin to the factored interaction surface
M_{OTM}	overturning moment due to factored actions
N	number of cycles to failure in fatigue of a specified constant amplitude stress range
Q_H	maximum horizontal foundation capacity
R	factored resistance
$R_{d,OTM}$	factored stabilizing moment
$R_{r,OTM}$	representative stabilizing moment
U	utilization
$U_{S,pl}$	utilization of preload

$U_{S,sl}$	utilization of foundation resistance to sliding
$U_{S,vhm}$	utilization of vertical and horizontal foundation capacity
V_{Lo}	maximum vertical reaction under the spudcan considered required to support the in-water weight of the jack-up during the entire preloading operation ⁴⁾
V_{st}	vertical reaction beneath the spudcan due to the assessment load case ⁵⁾
$W_{BF,A}$	submerged weight of the backfill that occurs after the maximum preload has been applied and held
$W_{BF,o}$	submerged weight of the overburden on top of the spudcan from backfill during preloading
$\gamma_{f,D}$	partial action factor applied to the inertial actions due to dynamic response
$\gamma_{f,E}$	partial action factor applied to the metocean or earthquake actions
$\gamma_{f,G}$	partial action factor applied to the actions due to fixed load
$\gamma_{f,V}$	partial action factor applied to the actions due to the variable load
$\gamma_{R,H}$	partial resistance factor for holding system strength
$\gamma_{R,Hfc}$	partial resistance factor for horizontal foundation capacity
$\gamma_{R,OTM}$	partial resistance factor for stabilizing moment
$\gamma_{R,PRE}$	partial resistance factor for preload
$\gamma_{R,S}$	partial resistance factor for spudcan strength
$\gamma_{R,VH}$	partial resistance factor for foundation capacity

4.2 Abbreviated terms

ALE	abnormal level earthquake
ALS	accidental limit states
BS	base shear
BSTF	base shear transfer function
CD	chart datum
DAF	dynamic amplification factor
ELE	extreme level earthquake
FE	finite element
FLS	fatigue limit states
IACS	International Association of Classification Societies
LAT	lowest astronomical tide

4) This is not the soil capacity. See definition 3.48.

5) See 8.8. Includes effects of leg weight and water buoyancy, but excludes effects of backfill and spudcan soil buoyancy.

LRFD	load and resistance factor design
LTB	lateral torsional buckling
MPM	most probable maximum
MPME	most probable maximum extreme
MSL	mean sea level
OCR	over-consolidation ratio
PDF	probability density function
PSIIP	project specific in-service inspection programme
RCS	recognized classification society
ROV	remotely operated vehicle
RPD	rack phase difference
SCF	stress concentration factor
SDOF	single degree-of-freedom
SLS	serviceability limit states
SWL	still water level
TRS	tropical revolving storm
ULS	ultimate limit states

5 Overall considerations

5.1 General

5.1.1 Competency

Assessments undertaken in accordance with this part of ISO 19905 shall be performed only 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 on which the assessment shall be based, following the general requirements specified in 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 part of ISO 19905. They form one whole and shall not be separated from one another.

For mobile offshore drilling units operating in regions subject to sea ice and icebergs, the requirements of this part of ISO 19905 shall be supplemented with the provisions relating to ice actions and procedures for ice

management contained in ISO 19906. The action factors from this part of ISO 19905 shall be applied. When joint probability data are not available, the factors from ISO 19906 may be used for the companion actions.

5.1.4 Reporting

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

5.1.5 Regulations

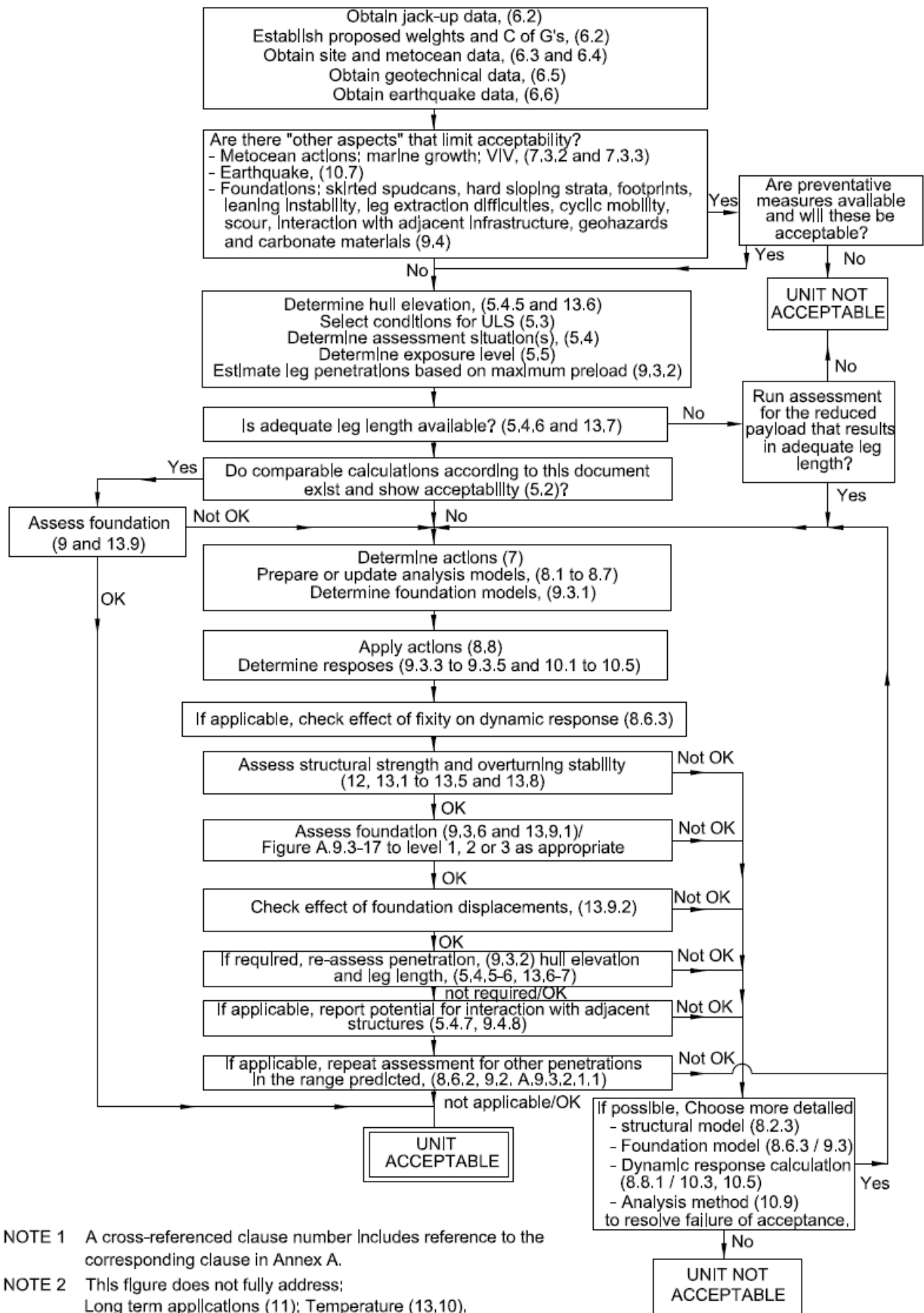
Each country can have its own set of regulations concerning offshore operations. It is the responsibility of the operator and jack-up owner to comply with relevant rules and regulations, depending upon the site and type of operations to be conducted.

5.2 Assessment approach

This subclause provides an overview of the data required, the assessment methodology, and the acceptance criteria. A flow chart for extreme storm assessment is shown for guidance in Figure 5.2-1. Annex A provides additional information and guidance, including detailed calculation methodology. Annex B specifies the partial factors for use in the assessment. Annexes C to F provide supplementary information or alternative calculation methodologies. Annex G provides a recommended contents list for the assessment report. The associated Technical Report, ISO/TR 19905-2, provides background to some of the recommendations given in the annexes and a detailed sample calculation. 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 part of ISO 19905.

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

- a) Compare assessment situations with design conditions or other existing assessments determined in accordance with this part of ISO 19905.
- b) Carry out appropriate calculations according to the simpler methods (e.g. pinned foundation, SDOF dynamics) given in this part of ISO 19905. 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 part of ISO 19905.



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

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

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

5.3 Selection of limit states

ISO 19900 divides the limit states into four categories as described below; normally, it is necessary to assess only the ULS in a jack-up site-specific assessment.

a) Ultimate limit states (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 required only in some areas of the world. The applicable partial action and resistance factors for the ULS and exposure level shall be as summarized in Annex B. For the ULS, the integrity of the structure should be unimpaired, but damage to the non safety-critical (secondary) structure of the jack-up can be tolerated.

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 in 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 states (SLS)

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

c) Fatigue limit states (FLS)

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

d) Accidental limit states (ALS)

The ALS are generally addressed at the design stage and it is not necessary to evaluate it in the assessment unless there are unusual risks at the site under consideration (e.g. when it is necessary 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 site (e.g. drilling mode/workover mode/tender mode/production mode). In each mode, the jack-up can be in the operating or storm survival configuration. Where more than one configuration is contemplated, the differences (e.g. the varying hull elevations required for each, skidding the cantilever in for a storm, reducing variable deck load) shall be considered in the assessment. The practicality of any required configuration change shall be evaluated and appropriate

assumptions incorporated into the assessment calculations. Any required restrictions on the operations shall be included in the operating procedures. The assessment situations shall be determined from appropriate combinations of mode, configuration and limit state.

Where the assessment indicates that an assessment situation does not meet the 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 his offshore personnel and, if relevant, to the operator. Alternatively, metocean data applicable to the season(s) of operation may be considered.

5.4.2 Reaction point and foundation fixity

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

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

5.4.3 Extreme storm event approach angle

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

5.4.4 Weights and centre of gravity

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

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

5.4.5 Hull elevation

The hull elevation used in the assessment shall 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 the prediction of penetration and to provide a contingency against settlement or scour. Leg length reserve requirements are given in 13.7.

5.4.7 Adjacent structures

The potential interaction of the jack-up with any adjacent structures shall be reported, as appropriate. Aspects requiring consideration by the operator include the effects of 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.

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

5.5 Exposure levels

5.5.1 General

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

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 respond 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 S2 or S3 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 can 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 site.

A jack-up shall always be considered S1 for the assessment 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. A jack-up may be categorized as S2 manned evacuated only if

- 1) reliable forecasting of an 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) documented plans are in place for obtaining forecasts and effecting evacuation prior to an extreme storm event; and
- 3) following the forecast of an extreme storm event, sufficient time and resources exist to safely evacuate all personnel from the jack-up (and any adjacent structure that can 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 manned only for occasional inspection, maintenance and modification visits. A jack-up may be categorized as S3 unmanned only 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 is often also referred to as “not normally manned”.

5.5.3 Consequence categories

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

- life-safety of personnel either on or near the jack-up who are brought in to respond 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 1 This classification includes risk of loss of human life for people other than personnel being part of the jack-up's normal complement and personnel on any adjacent structure that can be affected by failure of the jack-up. A primary driver for the classification in consequence categories is damage to the environment or to society (e.g. the situation where a community/state/country suffers significant losses as a consequence of the interruption of production). The classification is based on the assumption that the jack-up owner and the operator agree on the economic loss category to suit their tolerance of risk.

NOTE 2 The anticipated loss should reflect the cost of plugging and abandoning wells on damaged facilities.

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 site for there to be a high probability of impact if the jack-up collapses or drifts from site. They are unlikely to be “high consequence”, although they can have been designed to a higher categorization 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.

NOTE 2 Examples of high consequences include the potential for 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 site can be high consequence.

NOTE 4 All earthquake events are considered to be high consequence because of life-safety, see category S1 in 5.5.2 a).

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. A jack-up may be categorized as medium consequence only if all the following requirements are met.

- 1) All wells that can flow on their own in the event of structural or foundation failure are equipped with fully functional means of reliably closing in the well to prevent such flow, and such means shall be manufactured, tested and installed in accordance with applicable specifications.

The possibility of flow should be considered as a result of a 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 can be affected by failure of the jack-up are protected from releasing hydrocarbons, either by virtue of inventory and pressure regime or by check valves or sea floor 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 sites 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. A jack-up may be categorized as low consequence only if all the following requirements are met.

- 1) All wells that can flow on their own in the event of structural or foundation failure are equipped with fully functional means of reliably closing in the well to prevent such flow, and such means shall be manufactured, tested and installed in accordance with applicable specifications.

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

- 2) Oil storage is limited to process inventory.
- 3) Pipelines that can be affected by failure of the jack-up are 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 assessed as likely to cause low consequences to any facility it is operating over, or is adjacent to.

5.5.4 Determination of exposure level

The three categories each for life-safety and consequence can, in principle, be combined into nine exposure levels. However, the level to use 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 exposure levels are given in Table 5.5-1.

Table 5.5-1 — Determination of exposure level

Life-safety category	Consequence category		
	C1 High consequence	C2 Medium consequence	C3 Low consequence
S1 Manned non-evacuated	L1	L1	L1
S2 Manned evacuated	L1	L2	L2
S3 Unmanned	L1	L2	L3

The following provisions apply to categories L1, L2 and L3.

- L1: A manned or C1 jack-up shall be assessed for either the 50 year independent extremes with partial action factor of 1,15 or for the 100 year joint probability metocean data with partial action factor of 1,25 (see 8.8.1 and Annex B).
- 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 can be reached at the site prior to evacuation being effected (e.g. 50 year 48 hour notice sudden hurricane in tropical revolving storm areas). The assessment shall use the partial factors applicable to L1.

The unmanned post-evacuation case shall also be considered according to criteria to be agreed between the jack-up owner and the operator.

- L3: The unmanned, low-consequence (survivability) criteria, to be agreed between the jack-up owner and the operator.

For earthquake, a jack-up shall be assessed as L1 using a ULS screening check with a 1 000 year earthquake event.

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 tools, particularly structural analysis programs, consist of recognized commercially available software suites that, when used by experienced and well-trained operators, can be considered suitable for their standard areas of application. For these software suites, 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 appropriate 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 and algorithms.

6 Data to assemble for each site

6.1 Applicability

Clause 6 describes the data that are required to undertake an assessment. In this part of ISO 19905, the field is the general area where the jack-up is to operate; the site is the specific position/orientation within the field. The site data are normally a subset of the field data. The data that should be included in the assessment report are listed in Annex G, which can be used as a check list.

6.2 Jack-up data

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

- jack-up type;
- installed leg length;
- latest revision of the drawings, specifications and the operations manual;
- data pertaining to the strength, stiffness and operation of the leg-to-hull connection;
- proposed lightship and variable load and centres of gravity for each configuration, accounting for any changes that are not included in the latest revision of the operations manual;
- preloading capacity or pre-drive capability;
- limiting spudcan capacity, e.g. reactions and bearing pressure distribution(s) used in the design cases;
- design parameters including, where applicable, RPD limits and any proposed deviations for the intended operation;
- details of any relevant modifications.

6.3 Site and operational data

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

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

The assessor can use directional metocean data to optimize the jack-up heading. When directional metocean data are used in the assessment, the jack-up heading shall be specified.

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

6.4 Metocean data

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

- a) water depth (LAT or CD);
- b) tide and storm surge;

- c) wave data:
 - significant wave height and spectral peak period (stating whether intrinsic or apparent, as discussed in A.7.3.3.5),
 - maximum wave height and associated period (stating whether intrinsic or apparent, as discussed in A.7.3.3.5),
 - abnormal wave crest elevation (see A.6.4.2.4);
- d) current velocity and profile;
- e) wind speed and profile.

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

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

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

Directionality of wind, wave and current may be considered if accurate data are available. For deterministic analysis, wave kinematics factors may be applied to account for wave 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.

Either the 50 year return period of individual extremes or the 100 year return period of joint probability metocean data shall be used for the site-specific assessment of 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 provide consistent reliability levels, different action factors are used with actions determined for a 50 year return period of individual extremes and for a 100 year return period of joint probability metocean data.

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

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

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

6.5 Geophysical and geotechnical data

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

The field 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 can be sufficient to identify the location of the existing footprints, to assess the hazards associated therewith and refer to previous site data and preloading or penetration records; however, the accuracy of such information should be verified.

At sites where there is any uncertainty, borings/corings and/or in-situ testing (e.g. piezocone penetrometer tests) data are recommended at the planned site. Alternatively, the site can be tied-in to such data at another site by means of shallow seismic data. If data are not available prior to the arrival of the jack-up, it can be possible to take boring(s)/coring(s), etc., from the jack-up before preloading and jacking to full hull elevation. Suitable precautions should be taken to ensure the safety of the jack-up during this initial period on site and during subsequent preloading.

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

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

6.6 Earthquake data

Earthquake data shall be obtained through the use of ISO 19901-2. A jack-up shall always be assessed using exposure level L1 earthquake data.

7 Actions

7.1 Applicability

This clause presents an overview of, and basic requirements for, the modelling of actions for site-specific assessment in accordance with this part of ISO 19905.

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 this clause, and the corresponding A.7, 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 that shall be considered in general terms:

- a) metocean actions:
 - 1) actions on legs and other structures from wave and current, and
 - 2) actions on hull and exposed areas (e.g. legs) from wind;
- b) functional actions:
 - 1) fixed actions, and
 - 2) actions from variable load;
- c) indirect actions resulting from responses:
 - 1) displacement-dependent effects, and
 - 2) accelerations from dynamic response;
- d) earthquake actions;
- e) other actions.

7.3 Metocean actions

7.3.1 General

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

7.3.2 Hydrodynamic model

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

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

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

7.3.3 Wave and current actions

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

NOTE When waves are superimposed on a (uniform) current, the intrinsic reference frame for the waves travels at the speed and in the direction of the underlying current. An observer travelling at the same speed and in the same direction as the current is stationary with respect to the intrinsic reference frame and, therefore, measures the intrinsic wave period (see A.7.3.3.5 and ISO 19901-1:2005, 8.3 and A.8.3). The wave has only an intrinsic wave length; there is no apparent wave length.

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

Vortex induced vibration (VIV) is normally considered to be covered by class, but should be checked for jack-ups with large-diameter tubular legs when the current velocity exceeds that used in the design; see for example DNV-RP-C205^[7.3-1]; Grundmeier, Campbell and Wesselink^[7.3-2] and Blevins^[7.3-3].

7.3.4 Wind actions

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

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

The potential effects of wind-induced vortex induced vibration (VIV) should be considered, see for example DNV-RP-C205^[7.3-1]; Grundmeier, Campbell and Wesselink^[7.3-2] and Blevins^[7.3-3].

7.4 Functional actions

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

7.5 Displacement dependent effects

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

7.6 Dynamic effects

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

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-to-hull clearances and hull inclination shall be considered as described in 8.3.6 and 10.5.4.

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

8 Structural modelling

8.1 Applicability

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

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

8.2 Overall considerations

8.2.1 General

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

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

- suitable representation of the leg, leg-to-hull connection and the leg-foundation interaction, including non-linear effects as necessary;
- adequate detail to enable realistic assessment of the leg structure, the structural/mechanical components of the jacking and/or fixation system and the foundation.

8.2.2 Modelling philosophy

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

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

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

An explicit model of the conductor is rarely warranted.

8.2.3 Levels of FE modelling

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

- a) fully detailed model of all legs and leg-to-hull connections, with detailed or representative stiffness model of hull and spudcan;
- b) equivalent leg (stick model) and equivalent hull; equivalent stiffness model of all legs and spudcans, equivalent leg-to-hull connection springs and representative beam-element hull grillage;
- c) combined equivalent/detailed leg and hull; simplified lower legs and spudcans, detailed upper legs and leg-to-hull connections with detailed or representative stiffness model of the hull;
- d) detailed single leg (or leg section) and leg-to-hull connection model. This model shall be used in conjunction with the reactions at the spudcan or the forces and moments in the vicinity of the lower guide obtained from model b).

8.3 Modelling the leg

8.3.1 General

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

8.3.2 Detailed leg

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

8.3.3 Equivalent leg (stick model)

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

8.3.4 Combination of detailed and equivalent leg

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

8.3.5 Stiffness adjustment

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

8.3.6 Leg inclination

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

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

8.4 Modelling the hull

8.4.1 General

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

8.4.2 Detailed hull model

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

8.4.3 Equivalent hull model

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

8.5 Modelling the leg-to-hull connection

8.5.1 General

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

8.5.2 Guide systems

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

8.5.3 Elevating system

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

8.5.4 Fixation system

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

8.5.5 Shock pad – floating jacking systems

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

8.5.6 Jackcase and associated bracing

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

8.5.7 Equivalent leg-to-hull stiffness

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

8.6 Modelling the spudcan and foundation

8.6.1 Spudcan structure

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

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

8.6.2 Seabed reaction point

Selection of the reaction point shall be based on the 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-to-hull connection.

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

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

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;
 - hull structure;
 - spudcans.
- The non-structural mass shall include
 - hull equipment and outfitting;
 - mass of the variable load;
 - sea water supply system;
 - leg appurtenances;
 - marine growth;
 - entrapped water in flooded members and spudcans.
- Added mass shall include contributions from
 - submerged legs and leg components, e.g. chords and braces;
 - sea water caissons;
 - for earthquake assessments 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 non-linear behaviour is properly captured, as given in Equation (8.8-1):

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

where

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

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

- E_e are metocean actions due to the extreme storm event; see 8.8.4 ($E_e = 0$ for earthquake assessment);
- D_e is an equivalent set of inertial actions representing dynamic extreme storm effects or ground motion effects due to earthquakes; see 8.8.5 ($D_e = 0$ for stochastic storm assessment according to 10.5.3);
- D_e is an equivalent set of inertial actions induced by the ELE or ALE ground motion for earthquake assessment; see 8.8.8;
- γ are the partial action factors, as given in 8.8.1.2 to 8.8.1.4.

NOTE Reference can be made to Annex B, which contains all of the applicable factors for use 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 and A.10.5.2.2.3 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 actions due to the 50 year return period independent extreme metocean data;
- $\gamma_{f,E} = 1,25$ when applied to the actions due to the 100 year return period joint probability metocean data;
- $\gamma_{f,D} = 1,0$ and is applied to the inertial actions due to dynamic response.

8.8.1.3 Stochastic storm analysis

As discussed in A.10.5.3.2, in a stochastic storm analysis the metocean wind wave and current parameters are increased such that an action factor of 1,0 can be applied whilst achieving comparable global factored actions. Consequently 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 wind, wave and current metocean parameters, see 10.5.3, A.10.5.3;
- $\gamma_{f,D} = 1,0$ and is applied to the inertial actions due to dynamic response.

8.8.1.4 Earthquake analysis

8.8.1.4.1 The partial action factors for ELE analysis described in 10.7 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;
- $\gamma_{f,E} = 0,9$ when applied to the ELE actions;
- $\gamma_{f,D} = 1,0$ and is applied to the inertial actions induced by the ELE ground motion ($E_e = 0$).

8.8.1.4.2 The partial action factors for the ALE 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;
- $\gamma_{f,E} = 1,0$ when applied to the ALE actions;
- $\gamma_{f,D} = 1,0$ and is applied to the inertial actions induced by the ALE ground motion ($E_e = 0$).

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

8.8.2 Functional actions due to fixed load and variable load

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

- weight in air including appropriate solid ballast;
- weight of permanent enclosed liquid;
- buoyancy.

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

- weight of liquid and solid stores;
- applied drilling and conductor loads;
- weight of readily removable equipment.

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

8.8.3 Hull sagging

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

8.8.4 Metocean actions

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

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

8.8.5 Inertial actions

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

8.8.6 Large displacement effects

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

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

8.8.7 Conductor actions

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

8.8.8 Earthquake actions

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

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 part of ISO 19905 should always take precedence in case of conflict.

NOTE The foundations of mat-supported jack-ups are not specifically covered in this part of ISO 19905.

9.2 General

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

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

- the possible range of predicted leg penetrations;
- the possibility of rapid leg penetration and/or punch-through;
- likely scale of spudcan movements, e.g. due to consolidation, capacity exceedance;
- the effects of cyclic loading;
- the consequences of specific site conditions, such as are listed in 9.4.

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

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

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

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

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

- pinned model: simple pinned foundation for all legs;
- secant model: linear vertical, linear horizontal and secant rotational stiffness where the iterative reduction of rotational stiffness ensures compliance with the yield interaction surface;
- yield interaction model: non-linear vertical, horizontal and rotational stiffness model where the non-linear behaviour ensures compliance with the yield interaction surface;
- continuum model: non-linear continuum foundation model coupled to the structure; this model shall also account for the load-penetration behaviour beyond the penetration achieved by preloading.

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

9.3.2 Leg penetration during preloading

The 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 can 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. Cases that deviate from this assumption shall be assessed using appropriate methods. Uncertainties regarding the geotechnical data should be properly reflected in the interpretation and reporting of the analyses. For the special case of carbonate material, see 9.4.10 and A.9.4.10.

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

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

9.3.3 Yield interaction

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

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

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

9.3.4 Foundation stiffnesses

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

The reduction in stiffness as the spudcan reactions approach or exceed the yield surface shall be included in the analysis. There are different approaches to determining the softening of the stiffnesses. Where the reduction of stiffness is not included in the soil model, the provisions of A.9.3.4.2.3 should be used to determine the reduced rotational secant stiffness; the vertical and horizontal stiffness remain unchanged. The stiffness reduction is implicit in fully coupled yield interaction models and in non-linear continuum foundation models, as discussed in A.9.3.4.2.4 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 surface. Spudcan reactions resulting from responses based on a model with pinned foundations for all legs may be assessed using the simplified preload and windward leg checks, provided that the individual spudcan reactions satisfy the associated applicability requirements.

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

9.3.6 Acceptance checks

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

- a) Level 1: Preload and windward leg check with reactions from a response analysis based on a pinned spudcan model for all legs; Steps 1a and 1b shall both be completed for a Level 1 check:
 - Step 1a: Foundation capacity check of the leeward leg based on the preloading capability (A.9.3.6.2), and
 - Step 1b: Check of the windward leg (A.9.3.6.3).
- b) Level 2: Foundation capacity checks. One of the following three steps shall be completed for a Level 2 check:
 - Step 2a: Foundation capacity check and sliding resistance check (A.9.3.6.4), based on the vertical and horizontal reactions, assuming a pinned spudcan; or
 - Step 2b: Foundation capacity check and sliding resistance check (A.9.3.6.5), based on the vertical, horizontal and moment reactions from a spudcan model that includes rotational, vertical and horizontal foundation stiffness with rotational stiffness reduction; or
 - Step 2c: Foundation capacity check (A.9.3.6.5), based on the vertical, horizontal and moment reactions from a spudcan model that includes rotational, vertical and horizontal foundation stiffness with reduction of vertical, horizontal and rotational stiffnesses. A Level 3 displacement check shall be performed.
- c) Level 3: Displacement check (A.9.3.6.6). One of the following two steps shall be completed for a Level 3 check:
 - Step 3a: Simple check using the leg-penetration curve based on the results of a Level 2 check when the foundation capacity check fails and/or a check of the effects of windward leg sliding when the Level 2 sliding check fails; or
 - Step 3b: Numerical analysis of the complete jack-up and non-linear foundation coupled in vertical, horizontal and rotational degrees of freedom, e.g. finite element approach.

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

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

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

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

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

When a Level 2a or 2b assessment results in a foundation over-utilization, a Level 3 assessment can be used to calculate the associated displacements. For all Level 2c analyses, a Level 3a assessment shall be performed. The procedure shall account for the redistribution of forces resulting from the overload and displacement of the spudcan(s). The acceptability of structural utilizations, overturning utilizations, foundation utilizations and displacements shall be re-evaluated in accordance with the acceptance criteria in Clause 13. The resulting displacement of the jack-up shall neither lead to the possibility of contact with any adjacent structure nor exceed practical limitations for continued operations.

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

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

9.4 Other considerations

9.4.1 Skirted spudcans

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

- skirt penetration;
- filling of any voids within skirt should partial penetration occur;
- bearing capacity (which can exceed preload);
- settlement, including consolidation of trapped soils;
- moment capacity;
- sliding resistance;
- foundation stiffness;
- drainage paths;
- resistance to extraction;
- soil trapped within the skirt after extraction.

9.4.2 Hard sloping strata

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

9.4.3 Footprint considerations

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

The positioning of spudcans very close to, or partially overlapping, footprints shall be carefully considered. This is because of the difference in resistance between the original soil and the disturbed soil in the footprint area and/or the slope at the footprint perimeter. The resulting leg displacements and/or eccentric spudcan loading can cause damage to the jack-up. The situation can be complicated by the proximity of a fixed structure or wellhead. The interaction between a spudcan and a footprint is expected to be minimal when the edge-to-edge distance exceeds one spudcan diameter, see Stewart and Finnie^[A.9.4-15], Cassidy *et al.*^[A.9.4-16], Gaudin *et al.*^[A.9.4-20] and Gan *et al.*^[A.9.4-21].

9.4.4 Leaning instability

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

9.4.5 Leg extraction difficulties

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

9.4.6 Cyclic mobility

Cyclic loads can cause a progressive build-up of pore pressures within the foundation soils and consequent soil strength degradation (liquefaction). 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 pressures 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) that can result in the sea floor soils being eroded. The phenomenon of scour is observed around spudcans that are embedded in granular materials at sites with high sea floor flow velocities. If scour is recognized to potentially cause problems, then preventive measures shall be implemented. See A.9.4.7 for further guidance.

9.4.8 Spudcan interaction with adjacent infrastructure

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

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

9.4.9 Geohazards

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

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

9.4.10 Carbonate material

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

10 Structural response

10.1 Applicability

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

This clause presents methods for calculating the response of a jack-up including static and dynamic effects. This clause also presents a discussion of the important parameters affecting the dynamic response, including mass, stiffness and damping. Actions are presented in Clause 7. Stiffness and mass modelling, 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 that shall be checked to determine the adequacy of individual structural components;
- foundation reactions that shall be checked to determine foundation performance and global stability;
- displacements to check for interaction with adjacent structures.

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

10.3 Types of analyses and associated methods

A jack-up shall be assessed for the in-place elevated storm mode. 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 analysing a single combination of actions for each assessment situation;
- stochastic one-stage analysis in which extreme values of the responses of the jack-up are determined statistically by analysing multiple combinations of (environmental) actions for each assessment situation. Because of the inherent non-linearity of jack-ups, stochastic analyses are performed in the time domain;
- ultimate strength analysis in which the collapse strength of the jack-up structure and its foundation are determined.

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

Table 10.3-1 — Analysis requirements for different assessment situations

In-place elevated mode	Deterministic analysis		Stochastic analysis		Ultimate strength analysis
	Linear	Non-linear	Dynamic linear	Dynamic non-linear	
Ultimate and serviceability limit states (ULS and SLS)	See 10.5, A.10.5.2 and A.10.5.3				Generally outside the scope of this part of ISO 19905. See 10.9
Fatigue limit state (FLS)	See 10.6	not applicable	See 10.6	not applicable	not applicable
Accidental limit state (ALS)	Appropriate, but can be unduly conservative	Appropriate, but outside the scope of this part of ISO 19905	Appropriate, but can be conservative	Appropriate, but outside the scope of this part of ISO 19905	Generally outside the scope of this part of ISO 19905. See 10.8
Earthquake (ULS or ALS)	See 10.7 and A.10.7		Appropriate, but outside the scope of this part of ISO 19905		Generally outside the scope of this part of ISO 19905. See A.10.7.4

10.4 Common parameters

10.4.1 General

In 10.4 is presented 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-to-hull connection, foundation and the $P-\Delta$ geometric effects as defined by the modelling practices in Clause 8. A range of stiffness values should be considered if stiffness information is not well defined.

10.4.2.3 Mass

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

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

10.4.2.4 Variability in natural period

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

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

10.4.2.5 Cancellation and reinforcement

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

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

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

10.4.3 Damping

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

10.4.4 Foundations

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

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

10.4.5 Storm excitation

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

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

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

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

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

10.5 Storm analysis

10.5.1 General

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

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

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

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

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

10.5.2 Two-stage deterministic storm analysis

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

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

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

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

Table 10.5-1 — Methods of extreme storm analysis

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

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

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

A.10.5.2.2.3 gives load cases that should be considered when $K_{DAF,RANDOM}$ is used to determine the inertial loadset in a two-stage analysis. The first load case, Equation (A.10.5-4), is always required. When $(T_n/T_p) > 0,9$, additional load cases should be considered such as the three shown in A.10.5.2.2.3, Equations (A.10.5-5) to (A.10.5-7).

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

10.5.3 Stochastic storm analysis

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

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

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

The action factors on metocean actions for this analysis method 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 instead; see A.10.5.3.

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

10.5.4 Initial leg inclination

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

If the initial leg inclination is not explicitly modelled, the member forces and holding system forces from the analysis 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 A.10.5.4.

In all cases, the direction of the moment shall be such as to maximize the 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 the holding system. Hull strength and jackhouse to deck connections are considered to be covered by classification unless special circumstances apply;
- overturning stability and spudcan sliding;
- spudcan strength and foundation bearing capacity.

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

See also Clauses 9, 12 and 13.

10.6 Fatigue analysis

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

10.7 Earthquake analysis

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

- sites with the potential for cyclic mobility (e.g. liquefaction) (ISO 19901-2 site class F);
- sites with the potential for unacceptable additional leg penetrations if the preload reactions are exceeded (settlement limits can be reduced when operating adjacent to other structures);
- jack-ups where the ratio between the individual leg preload reaction at the 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 ULS for strength and stiffness, when it is subjected to earthquake actions derived from the uniform hazard spectrum for a return period of 1 000 years. Guidance on 1 000 year earthquake response spectrum criteria can be found in ISO 19901-2. In this kind of earthquake, the jack-up should sustain little or no damage.

If the jack-up does not satisfy this 1 000 year ELE screening to ULS assessment criteria, the alternative assessment methods (see 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 do not occur under any of the earthquake events considered, although in some cases considerable structural damage can 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 can be associated with greater lateral forces depending on the reduction in the transverse accelerations in combination with the increased mass.

The assessment model shall include a realistic range of spudcan-soil modelling that encompasses the uncertainties in foundation stiffness and capacities; see 8.6.3. Where the penetration predictions vary significantly, the range shall be considered. A pinned spudcan model, in general, produces an unconservative representation of the earthquake demand on the jack-up.

At sites 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. For manned situations, the acceptance criteria is typically set by the regulator. For unmanned/de-manned situations, the acceptance criteria shall be agreed between the operator and the jack-up owner. In some areas of the world, the analysis can entail

- assessing the jack-up for abnormal wave condition to demonstrate survivability (e.g. for a 10 000 year return period in the North Sea);
- scaling the extreme storm actions until failure is predicted to occur, to meet a target reserve strength ratio (e.g. Gulf of Mexico fixed structures; see ISO 19902:2007, 9.10.2);
- performing time-history analyses for the ALE (see ISO 19901-2).

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 in accordance with this part of ISO 19905.

11 Long-term applications

11.1 Applicability

When a jack-up is to be operated at one particular site for longer than the normal special survey period of five years, the site-specific assessment shall be supplemented by the provisions of Clause 11 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 intended to be on site;
- a list of modifications to the jack-up, which affect the time-varying actions, structural resistance or, fatigue endurance of structural components;

- the limitations on the ability to re-level the hull and maintain hull elevation, e.g. in connection with supported conductors;
- the deviations from the standard operating and elevated storm mode configurations given in the marine operations manual;
- the metocean data suitable for fatigue assessment, including directionality of wind, waves and current;
- the expected accumulation and vertical distribution of marine growth and relevant mitigation procedures;
- the geotechnical data required for the assessment of long-term operations;
- other data required for fatigue assessment (see 11.3.1).

11.3 Special requirements

11.3.1 Fatigue assessment

The remaining fatigue life of all relevant structural components shall be shown to be adequate for the planned period on site. In the assessment, any fatigue damage contributions from the jack-up's prior service shall be taken into account; historical jack-up and site data shall be requested from the jack-up owner. In view of the inherent uncertainty of fatigue life assessments, a margin of safety shall be applied through a fatigue damage design factor (f_{FD}). See A.11.3.1 for further details.

The partial action factors used for fatigue analysis can be reduced to unity when using S-N curves at mean minus two standard deviations of $\log(N)$.

11.3.2 Weight control

Changes in weight during the long-term operations shall be monitored to ensure 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 site. Special attention shall be given to corrosion protection in the splash zone.

11.3.4 Marine growth

The assessment shall include the effects of the long-term accumulation of marine growth.

11.3.5 Foundations

The assessment shall include consideration of the potential for and effects of

- settlement under extreme storm actions;
- long-term foundation settlement;
- seabed subsidence, e.g. due to reservoir depletion;
- scour;
- 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:

- a) a special survey prior to deployment on site;
- b) project specific surveys in accordance with an in-service inspection programme (PSIIP).

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

- RCS requirements;
- the jack-up's prior operating and inspection history;
- the assessment results for the expected operations.

Sea floor surveys shall be included in the PSIIP for sites where scour and/or seabed mobility are known to occur.

If changes to the initially planned duration are proposed by the operator, the jack-up owner should document that the jack-up has sufficient remaining fatigue life, and approval 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.

Equations for the required strength checks are given in this clause, which result in structural strength utilizations in accordance with Clause 13.

A suitable method for carrying out the required calculations is given in A.12. The resistance factors given in Annex B are specifically tied to the calculation methods presented in A.12 and shall be re-calibrated if other methods are used.

RCS requirements cover the design, construction, and periodic survey of the jack-up and address issues, such as material properties, fabrication tolerances, welding, construction details and parts of the jack-up other than the legs (e.g. jackhouse and hull structure) which are not normally addressed in a site-specific assessment. For example, when the forces within the fixation system are within the limits set by the manufacturer and are approved by the RCS, no additional assessment is required of the hull and jackhouse. Similarly, if the foundation's vertical and rotational reactions on the spudcan are within the structural limits set by the manufacturer, it is not necessary to check the strength of the leg to spudcan connection.

12.1.2 Truss type legs

The requirements set out in Clause 12 relate to chords and braces of truss type legs. Weld sizes, gusset plates, the strength of joints, etc., are covered by RCS requirements, and should not control the overall structural integrity. Chords and braces are covered in 12.2 to 12.6.

12.1.3 Other leg types

Some of the checks included in Clause 12 are applicable to either tubular or box-type legs, but for these configurations, Clause 12 should be supplemented with other documents to address stiffened sections, e.g. API references [12.1-1] and [12.1-2] or DNV references [12.1-3] and [12.1-4].

12.1.4 Fixation system and/or elevating system

Strength of the fixation system and/or the elevating system is normally supplied by the manufacturer. The manufacturer's data should represent the unfactored ultimate strength of the system(s) 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 manufacturer's data are expected to represent the unfactored ultimate strength of the spudcan and spudcan to leg connection, 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 member cross-sections (12.2);
- section properties of non-circular prismatic members (12.3);
- 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 in truss type legs (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 cross-sections of tubular members and for all other cross-sections of prismatic members. Longitudinally reinforced tubulars and tubulars with pin-holes, cut-outs, etc., shall be considered to be non-circular prismatic members.

12.2.2 Material yield strength

The material yield strength used in the member classification and the calculation of member strengths shall correspond to the value at 0,2 % strain offset from the initial linear stress-strain behaviour. A lesser value shall be used when the material does not exhibit sufficient work-hardening.

12.2.3 Classification definitions

The strength of a steel cross-section is affected by its potential to suffer local buckling when subjected to compression due to a bending moment or an axial force, or a combination thereof. By classifying cross-sections, the requirement to explicitly calculate local buckling strength is avoided.

For non-circular prismatic members, the components and cross-sections are classified as plastic, compact, non-compact (or semi-compact) and slender, in order of decreasing strength. When a cross-section is composed of components of different classes, it shall be classified according to the class of its component(s) with the lowest strength in compression. Slender components within a cross-section can be ignored, provided

that only the remaining cross-section is used for all aspects of the assessment. The following classification shall be applied.

- Class 1 Plastic: Cross-sections with plastic hinge rotation capacity. Compliance with this classification enables a plastic hinge to develop with sufficient rotation capacity to allow redistribution of moments to occur within the member. All plastic sections are inherently compact.
- Class 2 Compact: Cross-sections with plastic moment capacity. Compliance with this classification enables the full plastic moment capacity of a cross-section to be developed, but local buckling prevents the development of a plastic hinge with sufficient rotation capacity to permit plastic assessment.
- Class 3 Non-compact (or semi-compact): Cross-sections with between full yield moment capacity and plastic moment capacity. 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 capacity.
- Class 4 Slender: Cross-sections that buckle locally before the yield stress can be achieved. A cross-section is classified as slender if any of the compression components of the cross-section does not comply with the limits for non-compact components.

There is no requirement to classify tubular member cross-sections to the same extent as non-circular prismatic member cross-sections other than to identify those tubulars for which plastic hinge rotation capacity is possible (i.e. class 1). This is because the equations for tubular member cross-sections presented in A.12.5 account for local buckling, whether plastic or elastic.

12.3 Section properties of non-circular prismatic members

12.3.1 General

The requirements in 12.3 apply to rolled and welded non-circular prismatic members comprising one or more components, such as can be found in a chord section of a jack-up leg. Their cross-sectional properties shall be determined as described in 12.3.

Cross-sectional properties of tubular members are included within the determination of their strength and addressed in 12.5.

12.3.2 Plastic and compact sections

For class 1 plastic and class 2 compact sections, section properties can be determined assuming fully plastic properties.

Where elastic section properties are determined for class 1 and 2 sections instead of plastic section properties, these can be based on a fully effective cross-section and shall then be treated as for class 3 sections.

12.3.3 Semi-compact sections

Section properties for class 3 semi-compact sections shall be based on elastic properties assuming fully effective cross-sections. When considering a cross-section comprised of components having different yield strengths, the critical stress locations shall be evaluated as these do not necessarily coincide with the minimum section modulus or the principal axes.

The strength check is based on an interpolation between class 2 plastic capacity and class 3 elastic capacity.

NOTE The critical stress locations are typically at the edges of the components and are a function of the member forces, the yield strength of the component and its position within the cross-section of the member.

12.3.4 Slender sections

Cross-section properties for class 4 slender sections shall be determined using elastic principles. When the stress across the entire section is tensile, the full section may be used. If any part of the section is in compression, the sectional properties shall be reduced as required based on effective sections (see A.12.3.5).

12.3.5 Cross-section properties for the assessment

The nomenclature and selection of variables for use in the assessment of members are summarized in A.12.3.5.

12.4 Effects of axial force on bending moment

The moment resulting from the eccentricity between the elastic and plastic centroids of class 1, 2 and 3 sections shall be included in the assessment moment; this can occur in sections that include components of differing yield strengths. Similarly, for class 4 sections, there is an eccentricity between the full elastic centroid that is used in the structural response analysis and the centroid of the reduced section that is used in the member strength check. This moment correction shall be included for members in both tension and compression.

Euler moment amplification, or $p-\delta$ effects, shall be included for members in axial compression. When $p-\delta$ effects are not included in the structural response 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 can be determined using a rational analysis that includes joint flexibility and side-sway.

It is mentioned that, traditionally, the effects of Euler amplification are included in the strength checks. However, some analysis results implicitly include the effects of Euler amplification. The assessment should include the effects of both the global large displacement effects ($P-\Delta$) and the local member moment amplification ($p-\delta$). Large displacement effects ($P-\Delta$) 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 12.5 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 non-circular prismatic members

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

The requirements given in A.12.6 ignore the effects of hydrostatic pressure. The validity of this assumption shall be checked for all sealed non-circular prismatic members (see Figure A.12.6-1 and Table A.12.5-1).

12.7 Assessment of joints

Joint strength is normally addressed by the RCS for the metocean conditions given in the operations manual. If the assessor has concerns that the site conditions lead to joint loads that exceed those assessed by the RCS, joint strength shall be assessed.

13 Acceptance criteria

13.1 Applicability

13.1.1 General

This clause defines the criteria for checking the acceptability of a jack-up for operation at a specific site for the various limit states.

The partial action and resistance factors set out in the acceptance criteria have been developed in conjunction with the analysis methodology set out in the rest of this part of ISO 19905 and are valid only if used with this methodology. The factors do not necessarily provide adequate reliability if used with other methodologies.

The criteria for checking the acceptability of a jack-up include consideration of the following issues:

- structural strength of legs, spudcan, and holding system (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, foundation capacity, sliding displacement, settlement resulting from exceedance of the capacity envelope (13.9);
- interaction with adjacent infrastructure (13.10);
- temperature (13.11).

The assessment checks for structural strength, overturning stability and foundation integrity for each limit state and assessment situation are based on a utilization parameter as described in 13.2.

13.1.2 Ultimate limit states

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

The integrity of the foundation is central to the site-specific assessment of a jack-up.

Areas on jack-ups that are often critical with regard to structural strength are the legs at the lower guides, the legs between guides, the pinions and/or rack teeth, the fixation system and/or fixation system supports (if fixation system is fitted) and the leg to spudcan connection. Where there is a degree of foundation fixity, the lower parts of the leg shall be checked assuming an upper bound fixity value. Foundation fixity shall be included in the evaluation of the upper leg only when an applicable and detailed foundation study has been made.

Compliance in whole or in part can also be demonstrated through comparison with prior assessments conducted in accordance with the provisions of this part of ISO 19905.

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 five years.

13.2 General formulation of the assessment check

The assessment shall follow a partial safety factor format. The partial action factors shall be applied to actions, not the action effects. The partial resistance factors shall be applied to representative foundation capacities and structural strengths. When undertaking a stochastic time domain procedure that incorporates fully non-linear foundation responses, the MPME utilizations shall be calculated using the procedure set out in 10.5.3.

The **utilization** (see definition 3.78) for each limit state and assessment situation shall satisfy the requirement of Equation (13.2-1):

$$U \leq 1,0 \quad (13.2-1)$$

where U is the utilization to one significant decimal place.

For assessments where the relevant action effect can be expressed by a single response, U is of the general form:

$$U = \frac{\text{action effect due to factored actions}}{\text{factored resistance}} \quad (13.2-2)$$

For assessments where the relevant action effect consists of a combination of responses, the individual action effects and factored resistances combine into an interaction equation, I . In these cases the utilization, U , is equal to the value of I .

For assessments where the resistance is given by the yield interaction surface (for foundations) or the plastic interaction surface (for strength of non-circular prismatic members) the utilization is of the general form:

$$U = \frac{\text{length of the vector from a specified origin to the action effect}}{\text{length of the vector from the same origin to the factored interaction surface}} \quad (13.2-3)$$

Factored actions shall be determined in accordance with the assessment load case F_d in 8.8.

Action effects shall be determined in accordance with the requirements of Clauses 9, 10 and 12, and the associated guidance given in A.9, A.10 and A.12. The particular form of the utilization equation is determined by the foundation and strength checks formulated in these clauses.

Annex B summarizes the clause(s)/subclauses(s) in this part of ISO 19905 where the applicable calculation methodology and the associated assessment check(s) can be found, and lists the values of the partial action and resistance factors that shall be used.

NOTE Normally, both partial action and partial resistance factors are greater than unity: actions are multiplied by partial action factors and resistances are divided by partial resistance factors.

13.3 Leg strength assessment

The equations given in 13.2 shall be used to assess the utilization of the leg structure. The methodology for undertaking checks on the strength of members is described in Clause 12, together with the associated resistance factors.

13.4 Spudcan strength assessment

The forces on the top and bottom of the spudcan due to factored actions, for any of the applicable assessment situations, shall be checked against the factored ultimate strength derived from the manufacturer's specification using a partial resistance factor for spudcan strength of $\gamma_{R,S} = 1,15$. 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 performed with pinned spudcan support, the forces on the spudcan can be derived from the preload reaction and the soil ultimate moment strength.

13.5 Holding system strength assessment

The forces on the holding system due to factored actions, for any of the applicable assessment situations, shall be checked against the factored ultimate strength derived from the manufacturer's specification using a partial resistance factor for holding system strength of $\gamma_{R,H} = 1,15$. Where limited information is available, a rational approach shall be used.

13.6 Hull elevation assessment

A hull elevation resulting in at least 1,5 m clearance between the assessment return period extreme wave crest elevation and the underside of the hull shall be provided (see 6.4). The extreme wave crest elevation is normally determined from the extreme still water level (SWL) in A.6.4.4 and the wave crest elevation above SWL in A.6.4.2.4.

In some areas of the world an abnormal wave crest elevation (see A.6.4.2.4) that can affect the global response, can be greater than the extreme wave crest elevation plus 1,5 m. The hull elevation shall be sufficient to clear this abnormal wave crest elevation. Where appropriate metocean databases and reliability models exist, the abnormal wave crest elevation can be determined accounting for the joint probability of tide, surge and crest elevation.

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

NOTE 1 Metocean studies after hurricanes Katrina and Rita^[13.6-1] have suggested that there exist local wave crest enhancements with a small area of effect. When calculating the hull elevation for jack-ups, it is not necessary to consider these local effects over and above the abnormal crest elevation since they do not affect the jack-up globally.

NOTE 2 The air gap is defined in ISO 19900 as the clearance between the highest water surface that occurs during the extreme metocean conditions and the lowest exposed part not designed to withstand wave impingement. This differs from the definition historically used by the jack-up industry.

13.7 Leg length reserve assessment

The leg length reserve above the upper guides should account for the uncertainty in the prediction of leg penetration and account for any settlement. The leg length reserve shall be at least 1,5 m. The greater the uncertainty, the larger the leg length reserve that should be available. A larger reserve can also be required due to

- strength limitations of the top bay;
- the increase in the proportion of the leg bending moment carried by the holding system due to the effective reduction in leg stiffness at the upper guide;
- additional settlement due to scour;
- long-term foundation settlement;
- reservoir settlement.

13.8 Overturning stability assessment

The equations given in 13.2 shall be used to assess margin of safety against overturning of the jack-up. The utilization shall be calculated as the ratio of overturning moment due to the factored actions, M_{OTM} , and the factored stabilizing moment, $R_{d,OTM}$.

The overturning moment, M_{OTM} , shall be calculated about the overturning axis in the most critical assessment situation using the assessment load case F_d . For independent-leg jack-ups, the overturning axes shall pass through any two or more spudcan reaction points. The reaction points are described in 8.6.2 and A.8.6.2.

The factored stabilizing moment $R_{r,OTM}$ is calculated by Equation (13.8-1):

$$R_{d,OTM} = R_{r,OTM} / \gamma_{R,OTM} \quad (13.8-1)$$

where

$R_{r,OTM}$ is the representative stabilizing moment;

$\gamma_{R,OTM}$ is the partial resistance factor for stabilizing moment, $\gamma_{R,OTM} = 1,05$.

The representative stabilizing moment, $R_{r,OTM}$, shall be calculated for the same assessment situation and about the same axis as used for the calculation of the overturning moment and shall account for the following contributions:

- the stabilizing moment from fixed action with the jack-up at the displaced position resulting from the factored actions;
- the minimum stabilizing moment from the most onerous combination of minimum variable load and position of centre of gravity in accordance with 5.3, 5.4.4, 7.4 and A.7.4;
- the stabilizing moments provided by a degree of foundation fixity; any stabilizing moments from foundation fixity shall be calculated in accordance with Clause 9, taking account of any reduction of the moment fixity to comply with the yield surface of the foundation.

Large deflection (P- Δ) 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 increase in utilization 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.

NOTE The overturning check serves only the purpose of a traditional benchmark; the assessment is governed by the foundation checks.

13.9 Foundation integrity assessment

13.9.1 Foundation capacity check

The equations 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.

For a foundation integrity check at all levels, the preload utilization, $U_{S,pl}$, shall be computed and reported in accordance with A.9.3.6.2. The utilization shall satisfy Equation (13.9-1) or the alternative formulation of Equation (13.9-2):

$$U_{S,pl} = \frac{V_{st} + W_{BF,A}}{V_{Lo} / \gamma_{R,PRE}} \leq 1,0 \quad (13.9-1)$$

or

$$U_{S,pl} = \frac{F_V - W_{BF,o} + B_S}{V_{Lo} / \gamma_{R,PRE}} \leq 1,0 \quad (13.9-2)$$

where the symbols are as defined in A.9.3.

For a Level 2a check with pinned spudcans, the utilization of the vertical and horizontal foundation capacity, $U_{S,vhm}$, shall be determined in accordance with A.9.3.6.4.1 and shall satisfy Equation (13.9-3):

$$U_{S,vhm} = \frac{\text{length of vector 1}}{\text{length of vector 2}} \leq 1,0 \quad (13.9-3)$$

where the vectors are defined in A.9.3.6.4.1.

For a Level 2a check the utilization of the foundation resistance to sliding, $U_{S,sl}$, shall be computed in accordance with A.9.3.6.4.2 and shall satisfy Equation (13.9-4):

$$U_{S,sl} = \frac{\text{length of vector 1}}{\text{length of vector 2}} \leq 1,0 \quad (13.9-4)$$

where the vectors are defined in A.9.3.6.4.2.

For a Level 2b check with a degree of foundation fixity, the conditions of Equations (13.9-3) and (13.9-4) remain valid; see A.9.3.6.5.

In a Level 2c check, using a yield interaction or continuum foundation model, compliance with the foundation yield surface is inherently included and the above utilization checks are generally not performed. However, when sliding is not included in the model, a sliding check shall be undertaken in accordance with A.9.3.6.4.2 and Equation (13.9-4).

13.9.2 Displacement check

If the forces on any spudcan due to the assessment load case F_d result in a utilization, computed in accordance with 13.9.1, that exceeds 1,0, a further assessment may be performed as discussed in A.9.3.6.6. This assessment shall show that any additional settlements and/or the associated additional structural action effects are within acceptable limits. Furthermore, there shall be no operational limitations on levelling the hull and re-establishing a safe hull elevation, or alternatively safely departing the location.

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 Interaction with adjacent infrastructure

The displacement of the jack-up shall not

- lead to contact or adverse interaction with any adjacent structure;
- exceed practical limitations for continued operations.

13.11 Temperatures

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

NOTE The purpose of this check is to ensure that the field temperature is compatible with the material used in the jack-up construction.

Annex A (informative)

Additional information and guidance

NOTE The clauses/subclauses in this annex provide additional information and guidance on clauses/subclauses in the body of this part of ISO 19905. The same numbering system and heading titles have been used for ease in identifying the subclause in the body of this part of ISO 19905 to which it relates.

A.1 Scope

Although this part of ISO 19905 does not address the integrity of well conductors, The Institute for Petroleum provides guidance on their assessment; see Reference [A.1.1-1].

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 A.1

No guidance is offered.

A.4.2 Symbols used in A.2

No guidance is offered.

A.4.3 Symbols used in A.3

No guidance is offered.

A.4.4 Symbols used in A.4

No guidance is offered.

A.4.5 Symbols used in A.5

No guidance is offered.

A.4.6 Symbols used in A.6

D_1	directional spreading function as a function of n
D_2	directional spreading function as a function of s
D_3	directional spreading function as a function of σ
d	water depth
$F(\alpha)$	directionality function
f	wave frequency
H_{\max}	individual extreme wave height
H_s	increased significant wave height to account for wave asymmetry
H_{srp}	significant wave height for the assessment return period
h	reference depth for wind driven current
L	wave length of the wave with H_{\max} and T_{ass} in water depth d , according to the periodic wave theory used
N	inverse exponent of the power law wind profile
n	parameter exponent in D_1
S_y	smallest spacing between the legs of 3-legged jack-ups
$S_{\text{PM}}(\omega)$	Pierson-Moskowitz wave spectrum for a sea state
$S_{\text{JS}}(\omega)$	JONSWAP wave spectrum for a sea state
$S_{\eta\eta}(f)$	wave spectral density function expressed as a function of wave frequency
$S_{\eta\eta}(f, \alpha)$	directional short-crested power density spectrum
s	parameter in D_2
T_{ass}	intrinsic wave period associated with H_{\max}
T_p	apparent modal or peak period of the spectrum
$T_{p,i}$	intrinsic modal or peak period of the spectrum
$T_{z,i}$	intrinsic 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_{ref}	1 min sustained wind velocity at elevation Z_{ref} (normally at 10 m above MSL)
V_t	downwind component of mean spring tidal current
V_w	wind generated surface current
V_Z	the wind velocity at elevation Z above SWL under consideration
Z	elevation above SWL under consideration
z	vertical coordinate relative to SWL under consideration, positive upwards
Z_{ref}	reference elevation above MSL

α	angle between the direction of elementary wave trains and the dominant direction of the short-crested waves
γ	shape parameter of the peak enhancement factor in the JONSWAP spectrum
κ	kinematics reduction factor
ϕ	directional spreading factor based on latitude
σ	standard deviation of the normal distribution in D_3
ψ	latitude

A.4.7 Symbols used in A.7

A	area
A_e	effective area of leg per unit height
A_i	effective area of member or gusset i
A_{Wi}	projected area of the block i perpendicular to the wind direction
C_A	added mass coefficient
C_{De}	equivalent value of the drag coefficient of a leg bay
C_{Dei}	equivalent value of the drag coefficient of member i
C_D, C_{Di}	drag coefficient, drag coefficient of member i
$C_{Dpr}(\theta)$	drag coefficient related to the projected diameter
C_{D0}	drag coefficient for a tubular with appropriate roughness
C_{D1}	drag coefficient for flow normal to the rack related to projected diameter, W
C_M, C_{Mi}	inertia coefficient, inertia coefficient of member i
C_{Me}	equivalent value of the inertia coefficient of a leg bay
C_{Mei}	equivalent value of the inertia coefficient of member i
C_s	shape coefficient
D, D_i	reference diameter, reference diameter of member i
D_e	equivalent diameter of leg
D_F	face width of leg, outside dimensions, orthogonal to the flow direction
$D_{pr}(\theta)$	projected diameter
d	water depth
H_s	increased significant wave height to account for wave asymmetry
l_i	length of member i node to node centre
m_a	added mass contribution (per unit length) for a member
P_i	pressure at the centre of block i
s	height of one bay, or part of bay considered

T_i	intrinsic period of a periodic wave (in a reference frame that is stationary with respect to the wave, i.e. with no current present)
T_n	first natural period of surge or sway motion of the jack-up
T_p	apparent modal or peak period of the spectrum
$T_{p,i}$	intrinsic modal or peak period of the spectrum
T_z	apparent mean zero-crossing period of the water surface elevation in a sea state
$T_{z,i}$	intrinsic mean zero-crossing period of the water surface elevation in a sea state
t_m	marine growth thickness
W	projected width
\dot{r}_n	velocity of the considered member, normal to the member axis and in the direction of the combined particle velocity
\ddot{r}_n	acceleration of the considered member, normal to the member axis and in the direction of the combined particle velocity
u	wave particle velocity
u_n	wave particle velocity resolved normal to the member axis
\dot{u}_n	wave particle acceleration resolved normal to the member axis
V_C	current velocity for use in the hydrodynamic model
V_f	far field (undisturbed) current velocity
$V_{z,i}$	wind velocity at the centre of block i
v_n	fluid particle velocity resolved normal to the member axis
z'	modified coordinate for use in particle velocity formulation
z	vertical coordinate relative to SWL under consideration, positive upwards, at which the kinematics are required
α_i	angle between flow direction and member axis projected onto a horizontal plane
β_i	angle defining the member inclination from horizontal
ΔF	wave action per unit length
ΔF_{drag}	drag action per unit length
$\Delta F_{\text{inertia}}$	inertia action per unit length
λ	wave length
ρ	mass density of water or air
θ	angle in degrees
ζ	instantaneous water level (same axis system as z)

A.4.8 Symbols used in A.8

A	axial area of equivalent leg model
A_s	effective shear area
E	Young's modulus of steel
F	applied axial action
G	shear modulus
I	second moment of area
K_{hh}	horizontal leg-to-hull connection stiffness
K_{rh}	rotational leg-to-hull connection stiffness
K_{vh}	vertical leg-to-hull connection stiffness
L	cantilevered length (from the hull to the seabed reaction point)
M	applied moment
P	applied shear
P_g	sum of the leg forces due to functional actions on legs at hull, including the weight of the legs above the hull
Δ	axial deflection (shortening) of the leg at the point of force application from the detailed leg model
Δ_C	axial end displacements of the combined leg and leg-to-hull connection model
δ	lateral deflection of the cantilevered leg at the point of moment application from the detailed leg model
δ_C	lateral deflection of the combined leg and leg-to-hull connection model
θ_C	slope of the end of the cantilever from the combined leg and leg-to-hull connection model
θ_M	slope of the cantilever at the point of moment application from the detailed leg model
θ_P	slope of the cantilever at the point of shear application from the detailed leg model

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
a_s	bearing capacity squeezing factor constant
B	effective spudcan diameter at uppermost part of bearing area in contact with the soil (for rectangular footing B equal to width)
B_{max}	diameter of the contact area in plan when the spudcan is fully seated
B_S	soil buoyancy of spudcan below bearing area i.e. the submerged weight of soil displaced by the spudcan below D , the greatest depth of maximum cross-sectional spudcan bearing area below the sea floor
b_s	bearing capacity squeezing factor constant dependent on spudcan diameter

C_H	horizontal capacity coefficient
D	greatest depth of maximum cross-sectional spudcan bearing area below the sea floor (see Figure A.9.3-3)
D_b	depth of backflow; infill should not be considered
D_R	relative density of sand (percent)
d	depth beneath sea floor
d_c	bearing capacity depth factor
d_{crit}	depth at which maximum bearing resistance occurs (layered case)
d_q	depth factor for drained soils
d_γ	depth factor on surcharge for drained soils
F_H	horizontal force applied to the spudcan due to the assessment load case (see 8.8)
F_M	moment force applied to the spudcan due to the assessment load case (see 8.8)
F_V	gross vertical force acting on the soil beneath the spudcan due to the assessment load case F_d (see 8.8)
$(F_V/Q_V)_t$	vertical load at intersection of adhesion yield surface and foundation yield surface
f_1	factor 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	foundation rotational stiffness reduction factor
G	shear modulus of the foundation soil
H	distance from spudcan maximum bearing area to weaker layer below
H_{cav}	limiting depth of cavity that remains open above the spudcan during penetration
h_1	embedment depth to the uppermost part of the spudcan, (if not fully embedded, $h_1 = 0$)
h_2	spudcan tip embedment depth
I_{rNC}	rigidity index for normally consolidated clays
I_P	plasticity index
j	dimensionless stiffness factor
k_a	active earth pressure coefficient (for $s_u = 0$)
k_p	passive earth pressure coefficient
K_1, K_2, K_3	stiffness factors for vertical, horizontal and rotational foundation stiffness respectively
K_{d1}, K_{d2}, K_{d3}	depth factors for vertical, horizontal and rotational foundation stiffness respectively
K_s	coefficient of punching shear
L_s	length of strip footing
m	parameter to define effect of adhesion on the foundation yield surface envelope
n_s	load spread factor for sand overlying clay
N_c	bearing capacity factor, taken as $N_c s_c = 6,0$ for circular footings

N_q	bearing capacity factor for a flat rough circular footing
N_γ	bearing capacity factor for a flat rough circular footing
p_o'	effective overburden pressure at depth, D , of maximum bearing area
p_a	atmospheric pressure
Q_0	spudcan bearing capacity at sea floor
Q_H	maximum horizontal foundation capacity
Q_{Hs}	foundation sliding capacity
Q_M	ultimate moment capacity of foundation
Q_{Mp}	increased ultimate moment capacity due to further spudcan penetration under environmental actions
Q_{Mps}	ultimate moment capacity when further spudcan penetration leads to full contact of the entire underside of the spudcan with the seabed
Q_{Mpv}	ultimate moment capacity under further spudcan penetration, when the applied vertical force is too low to achieve full contact of the entire underside of the spudcan with the seabed
Q_{peak}	maximum bearing capacity at $d = d_{crit}$
$Q_{u,b}$	ultimate vertical foundation bearing capacity assuming the spudcan bears on the surface of the lower (bottom) clay layer with no backfill
Q_V	gross ultimate vertical foundation capacity
Q_{Vnet}	net ultimate vertical foundation capacity
Q_{Vo}	initial gross ultimate vertical foundation capacity established by preload operations
r_f	failure ratio
R_{OC}	over-consolidation ratio
s_c	bearing capacity shape factor
s_u	undrained shear strength
$s_{u,a}$	undrained shear strength of backfill material above the spudcan
$s_{u,o}$	undrained shear strength at deepest depth of maximum bearing area (D below sea floor)
s_{uH}	undrained shear strength at depth of H_{cav} below sea floor
$s_{u,l}$	undrained shear strength at the spudcan tip
s_{um}	undrained shear strength at the sea floor
$s_{u,b}$	undrained shear strength of lower clay layer below spudcan
$s_{u,t}$	undrained shear strength of upper clay layer below spudcan
T	thickness of weak clay layer underneath spudcan
V_D	volume of the spudcan below the maximum bearing area that is penetrated into the soil
V_L	available spudcan reaction
V_{Lo}	maximum vertical reaction under the spudcan considered required to support the in-water weight of the jack-up during the entire preloading operation (this is not the soil capacity; see 3.48)

V_{st}	vertical reaction beneath the spudcan due to the assessment load case, see 8.8, (includes effects of leg weight and water buoyancy but excludes effects of backfill and spudcan soil buoyancy)
V_{spud}	the total volume of the spudcan beneath the backfill
V_{sw}	gross vertical spudcan reaction under still water conditions for the spudcan being considered (includes effects of backfill and spudcan soil buoyancy)
W_{BF}	submerged weight of the backfill
$W_{BF,A}$	submerged weight of the backfill that occurs after the maximum preload has been applied and held
$W_{BF,o}$	submerged weight of the backfill during preloading
$W_{BF,omin}$	minimum value of the submerged weight of the backfill, due to backflow during preloading
α	adhesion factor
β	equivalent cone angle
δ	steel/soil friction angle in degrees
$\gamma_{R, Hfc}$	partial resistance factor for horizontal foundation capacity
$\gamma_{R, VH}$	partial resistance factor for foundation capacity
γ'	submerged (effective) unit weight of soil
ρ	rate of increase in undrained shear strength with depth
ϕ'	effective angle of internal friction for sand in degrees
ν	Poisson's ratio

A.4.10 Symbols used in A.10

B	equivalent spudcan diameter at uppermost part of bearing area in contact with the soil
C_{rd}	radiation damping coefficient of a dashpot (force per unit velocity)
D_e	equivalent set of inertial actions representing dynamic extreme storm effects or ground motion effects due to earthquakes
E_e	metocean actions due to the extreme storm event
$F_{BS,Amplitude}$	single amplitude of quasi-static base shear over one wave cycle
$F_{BS,(QS)Max}$	maximum quasi-static wave/current base shear
$F_{BS,(QS)Min}$	minimum quasi-static wave/current base shear
F_{in}	magnitude of the inertial loadset
G	shear modulus
G_F	actions due to the fixed load positioned such as to adequately represent their vertical and horizontal distribution
G_V	actions due to maximum or minimum variable load, as appropriate, positioned at the most onerous centre of gravity location applicable to the configurations under consideration
K_{eff}	effective system stiffness

$K_{\text{DAF,RANDOM}}$	DAF from random wave time domain (stochastic) analyses
$K_{\text{DAF,SDOF}}$	DAF from single degree-of-freedom representation of dynamic behaviour
M_{eff}	effective system mass
O_{T}	total horizontal offset of the leg base with respect to the hull
O_1	offset due to leg-to-hull clearances
O_2	offset due to maximum hull inclination permitted by the operating manual
T_{n}	first natural period of surge or sway motion of the jack-up
T_{p}	apparent modal or peak period of the wave spectrum
$T_{\text{p,i}}$	intrinsic modal or peak period of the wave spectrum
ν	Poisson's ratio of the foundation soil
Ω	ratio of jack-up natural period to wave excitation period
ρ	total, saturated, (mass) density of the foundation soil
ζ	damping ratio or fraction of critical damping
ζ_{rd}	radiation modal damping ratio to account for spudcan vertical motion
ω_{n}	natural frequency (rad/s)

A.4.11 Symbols used in A.11

$D_{\text{c,e}}$	calculated existing fatigue damage prior to arriving at site
$D_{\text{c,s}}$	calculated fatigue damage during planned operations on site
$f_{\text{FD,e}}$	fatigue damage design factor applicable to $D_{\text{c,e}}$
$f_{\text{FD,s}}$	fatigue damage design factor applicable to $D_{\text{c,s}}$
N	number of cycles to failure in fatigue of a specified constant amplitude stress range, S
S	constant amplitude stress range

A.4.12 Symbols used in A.12

A	gross cross-sectional area
A_{ec}	total effective area of a slender section in compression of a non-circular prismatic member
A_{c}	cross-sectional area for use in the assessment of a non-circular prismatic member in compression
$A_{\text{eff,i}}$	effective area of a component of a non-circular prismatic member in compression
A_{f}	cross-sectional area of a semi-compact section of a non-circular prismatic member
A_i	cross-sectional area of the i th component comprising the structural member
A_{o}	the area enclosed by the median line of the perimeter material of a section
A_{p}	fully plastic effective cross-sectional area of a non-circular prismatic member
A_{t}	cross-sectional area for use in the assessment of a non-circular prismatic member in tension

A_v	effective shear area of a non-circular prismatic member in the direction being considered
B	member moment amplification factor for the axis under consideration
B_s	overall breadth of cross-section
b_w	width of the wall of a component forming the closed perimeter of a section
b	effective width of a component
b_1	width of base plate
b_2	width of reinforcing plate
C_m	moment reduction factor
C_x	critical elastic buckling coefficient
D	outside diameter of a tubular
D_s	overall depth of cross-section
d	effective depth of a component
d_w	effective head of water
d_i	distance between the centroid of the i th component and the plastic neutral axis
E	Young's modulus of steel (elastic modulus)
e	eccentricity between the axis used for structural analysis and that used for structural strength checks
e_a	effective eccentricity between the axis used for structural analysis and that used for structural strength checks for class 3 members
F_{cr}	reduced material strength
F_y	yield strength in stress units
F_{yeff}	effective yield strength of the cross-section of a non-circular prismatic member in stress units
F_{yi}	yield strength of the i th component of the cross-section of a non-circular prismatic member in stress units
F_{ymin}	minimum yield strength of all components in the cross-section of a non-circular prismatic member (minimum value of F_{yi} , in stress units)
$F_{y,ltb}$	yield strength, F_y of the material that first yields when bending about the minor axis
g	acceleration due to gravity
h	subscript referring to the component that produces the smallest value of P_{pl}
I	second moment of area
I_e	effective second moment of area of a non-circular prismatic member cross-section
I_f	second moment of area of a plastic, a compact or a semi-compact section of a non-circular prismatic member cross-section
I_p	polar moment of inertia of a tubular
I_{pp}	polar moment of inertia a non-circular prismatic member
I_1	major axis second moment of area of the gross cross-section

I_2	minor axis second moment of area of the gross cross-section
J	torsion constant
K	effective length factor
L	unbraced length of member for the plane of flexural buckling
L_b	effective length of a beam-column between supports
L_p	limiting plastic length
L_r	limiting unbraced length for inelastic torsional buckling
M_b	representative bending moment strength of a tubular or a non-circular prismatic member
M_{by}, M_{bz}	representative bending moment strength about member y- and z-axes, respectively
M_p	plastic moment strength of a tubular or a non-circular prismatic member
M_{py}, M_{pz}	plastic moment strengths of a tubular or a non-circular prismatic member about member y- and z-axes, respectively
M_u	bending moment in a member due to factored actions determined in an analysis that includes global P- Δ effects
M_{ua}	amplified bending moment M_u
M_{ue}	corrected effective bending moment M_u
M_{uay}, M_{uaz}	amplified bending moments due to factored actions about member y- and z-axes, respectively
M_{uey}, M_{uez}	corrected bending moments due to factored actions about member y- and z-axes, respectively
M_{uy}, M_{uz}	bending moments due to factored actions about member y- and z-axes, respectively, determined in an analysis that includes global P- Δ effects
P_a	representative axial compressive strength of a tubular
P_E	Euler buckling capacity
P_n	representative axial compressive strength based on local strength for column buckling of a non-circular prismatic member
P_p	representative axial strength of a non-circular prismatic member
P_{pl}	representative local axial compressive strength of non-circular prismatic member prismatic members
P_t	representative axial tensile strength of a non-circular prismatic member
P_u	axial force in a member due to factored actions determined in an analysis that includes global P- Δ effects
P_{ut}	axial tensile force due to factored actions
P_{uc}	axial compressive force due to factored actions
P_v	representative shear strength of a tubular
P_{vy}, P_{vz}	are the representative shear strengths in the local y- and z-directions of a non-circular prismatic member, respectively
P_{xe}	representative elastic local buckling strength of a tubular
P_y	plastic strength of a non-circular prismatic member

P_{yc}	representative local buckling strength of a tubular
p	depth below sea floor (zero if above sea floor)
r_{ltb}	radius of gyration about the minor axis when used for lateral-torsional buckling considerations
r	radius of gyration for the plane of flexural bending
r_t	maximum distance from centroid to an extreme fibre for torsional shear check
S_e	reduced effective section modulus of a slender section of a non-circular prismatic member
S_f	elastic section modulus of a semi-compact section of a non-circular prismatic member
S_y, S_z	section moduli for use in the assessment of a non-circular prismatic member in flexure
T_u	torsional moment due to factored actions
T_v	representative torsional strength of a tubular
t	wall thickness of a tubular
t_1	thickness of base plate
t_2	thickness of reinforcing plate
t_f	thickness of a flange component
t_w	thickness of a web component
V	beam shear due to factored actions
V_y, V_z	beam shears due to factored actions in the local y- and z-directions, respectively
y_i	distance from the neutral axis associated with I_e to the critical point i
Z_p	fully plastic (effective) section modulus
α	factor that varies depending on the type of loading
γ'	submerged (effective) unit weight of soil
$\gamma_{R,Pa}$	partial resistance factor for axial strength of a non-circular prismatic member
$\gamma_{R,Pb}$	partial resistance factor for bending strength of a non-circular prismatic member
$\gamma_{R,Pcl}$	partial resistance factor for local axial compressive strength of a non-circular prismatic member
$\gamma_{R,Pt}$	partial resistance factor for axial tensile strength of a non-circular prismatic member
$\gamma_{R,Pc}$	partial resistance factor for axial compressive strength of a non-circular prismatic member
$\gamma_{R,Pv}$	partial resistance factor for torsional and beam shear strength of a non-circular prismatic member
$\gamma_{R,Tb}$	partial resistance factor for bending strength of a tubular
$\gamma_{R,Tt}$	partial resistance factor for axial tensile strength of a tubular
$\gamma_{R,Tc}$	partial resistance factor for axial compressive strength of a tubular
$\gamma_{R,Tv}$	partial resistance factor for torsional and beam shear strength of a tubular
k	buckling coefficient
λ	column slenderness parameter
λ_h	ratio b/t or $2R/t$ as applicable for component h

λ_c	prismatic column slenderness parameter for a non-circular prismatic member
λ_r	elastic plate slenderness parameter
λ_p	plastic plate slenderness parameter
λ_{plim}	limiting plate slenderness ratio
λ_{po}	plate slenderness ratio coefficient
η	exponent for biaxial bending, a constant dependent on the prismatic member cross-section geometry
ρ	reduction coefficient
ρ_w	mass density of water
σ_1	compressive stress if σ_2 tensile or the larger compressive stress if σ_2 is also compressive
σ_2	tensile stress if σ_2 tensile or the smaller compressive stress if σ_2 is compressive
ψ	ratio of compression to bending stress

A.5 Overall considerations

No guidance is offered.

A.6 Data assembled for each site

A.6.1 Scope

No guidance is offered.

A.6.2 Jack-up data

No guidance is offered.

A.6.3 Site data

No guidance is offered.

A.6.4 Metocean data

A.6.4.1 General

The jack-up should be assessed for the extreme storm event (ULS assessment). For manned jack-ups (category S1) the 50 year return period independent extremes should be used. Alternatively, 100 year joint probability metocean data may be used. The action factors for these two alternatives differ.

If the jack-up life safety category is manned evacuated, it is assumed that reliable forecasting of the extreme storm event is feasible, that evacuation plans are established and documented, and that time and resources are available to safely evacuate all personnel from the jack-up and any adjacent structures that can be affected by failure of the jack-up (see 5.5). Under these conditions, hindcast storm characteristics may be computed based on the threshold time horizon of storm formation relative to the jack-up site. The time horizon is defined as the time required for safe evacuation, and the extreme storm event is derived from the population of storms that can develop and impact the jack-up site within that time horizon.

A “sudden hurricane” is defined as one that forms locally and, due to speed of formation and proximity to infrastructure at time of formation, might not allow sufficient time to evacuate manned facilities. The population of storms used to derive the sudden hurricane at a given site can therefore be defined in terms of the time horizon required to evacuate the site. For manned evacuated jack-ups utilized in these circumstances, consideration should be given to the use of a 50 year return period “sudden hurricane”. 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 do generally not take account of local variations.

Where the actions due to metocean conditions at the site are directional, the jack-up may be aligned on an advantageous heading subject to practical and infrastructure limitations at the site.

A.6.4.2 Waves

A.6.4.2.1 General

The extreme wave environment should be determined in accordance with A.6.4.2.2 to A.6.4.2.10. It should be based on the three hour storm exposure for the relevant assessment return period (e.g. 50 year independent extremes or 100 year joint probability). The seasonally adjusted wave height may be used when appropriate for the proposed operation. When a fatigue analysis is required (see Clause 11), long-term wave data should be obtained.

The assessor should check the consistency of the wave data provided, giving particular attention to the wave periods and the ratio of H_{\max} to H_{srp} and query any apparent inconsistencies with the data provider.

A.6.4.2.2 Extreme wave height

The wave height information for a specific site can be expressed in terms of H_{\max} , the individual extreme wave height for the assessment return period, or the significant wave height H_{srp} . The relationship between H_{srp} and H_{\max} should be determined accounting for the duration of a storm (three hours minimum) and for the additional probability of other return period storms; see ISO/TR 19905-2:2012, 6.4.2.2. This relationship depends on the regional and site-specific conditions, however H_{srp} may usually be determined from H_{\max} using the generally accepted relationship for non-cyclonic areas as given in Equation (A.6.4-1):

$$H_{\max} = 1,86 H_{\text{srp}} \quad (\text{A.6.4-1})$$

For cyclonic areas the recommended relationship is as given in Equation (A.6.4-2):

$$H_{\max} = 1,75 H_{\text{srp}} \quad (\text{A.6.4-2})$$

The wave action can be computed deterministically (through an individual maximum wave approach) or probabilistically (through a time domain simulation). The two methods are discussed in A.6.4.2.3 and in A.6.4.2.5 to A.6.4.2.8, respectively (see also ISO/TR 19905-2:2012, 6.4.2). The two methods should be used in conjunction with the associated kinematics modelling recommended in A.7.3.

A.6.4.2.3 Deterministic waves

For the calculation of wave actions using a deterministic (regular) wave, it is appropriate to apply a kinematics reduction factor to the horizontal and vertical velocities and accelerations in order to obtain realistic estimates of the actions for the extreme storm event. This factor ensures that both the deterministic (regular) calculation of wave action using a regular wave and the three-hour stochastic simulation produce statistically comparable results (i.e. both target the MPME response in the 50 year extreme storm event). In addition, the factor takes some account of wave spreading and the conservatism of regular wave kinematics. The kinematics reduction factor can be applied either by scaling of wave kinematics (preferred) or by a wave height reduction, but not both.

The kinematics reduction factor, κ , to be applied to the kinematics obtained from H_{\max} can be determined from Equation (A.6.4-3):

$$\kappa = \phi \quad (\text{A.6.4-3})$$

where

ϕ is the directional spreading factor defined in ISO 19901-1:2005, A.8.7.2, for the site-specific metocean data or for open water conditions; it is based on the latitude ψ in degrees and the type of storm or region:

for low latitude monsoons with typically $|\psi| < 15^\circ$ $\phi = 0,88$

for tropical cyclones below approximately 40° latitude $\phi = 0,87$

for extratropical storms for the range of latitudes $36^\circ < |\psi| < 72^\circ$ $\phi = 1,019\ 3 - 0,002\ 08 |\psi|$.

Alternatively, Equations (A.6.4-4) to (A.6.4-7) can be used; see Reference [A.6.4-1]:

$$\begin{aligned} \kappa = & 0,824\phi + 0,426\phi^2 - 0,043\left(\frac{S_y}{L}\right)\phi - 1,450\left(\frac{S_y}{L}\right)^2\phi - 0,800\left(\frac{d}{L}\right)\phi + \dots \\ & \dots + 0,658\left(\frac{d}{L}\right)^2 - 0,640\left(\frac{H_{\max}}{d}\right) + 1,303\left(\frac{H_{\max}}{d}\right)^2\phi^2 \end{aligned} \quad (\text{A.6.4-4})$$

and subject to the following:

$$0,08 \leq \left(\frac{S_y}{L}\right) \leq 0,43 \quad (\text{A.6.4-5})$$

$$0,14 \leq \left(\frac{d}{L}\right) \leq 0,76 \quad (\text{A.6.4-6})$$

$$0,07 \leq \left(\frac{H_{\max}}{d}\right) \leq 0,58 \quad (\text{A.6.4-7})$$

where

S_y is the smallest spacing between the legs of 3-legged jack-ups;

d is the water depth;

H_{\max} is the maximum wave height;

T_{ass} is the intrinsic wave period associated with H_{\max} ;

L is the wave length of the wave with H_{\max} and T_{ass} in water depth d , according to the periodic wave theory that is being used.

The limiting values $\frac{S_y}{L} = 0,43$, $\frac{d}{L} = 0,76$ and $\frac{H_{\max}}{d} = 0,07$ may be applied for calculation of κ in Equation (A.6.4-4) in case these bounds are transgressed. In all cases, it is not necessary that κ be greater than ϕ .

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

In lieu of using the kinematics reduction factor, the effects of wave spreading can be explicitly included in the analysis method, provided that higher frequency interaction effects (e.g. those due to frequency sum terms) are appropriately modelled through the use of second (or higher) order wave theory. Frequency interaction effects introduce additional actions that offset some of the reduction in actions predicted by three-dimensional linear wave theories. See A.7.3.3.3.2.

The wave actions should be determined using an appropriate wave kinematics model in accordance with A.7.3.3.1.

In the analysis, a single value for the intrinsic wave period T_{ass} , expressed in seconds, associated with the maximum wave can be used. The “intrinsic” period of the wave as seen by an observer moving with the current should be used in the derivation of wave kinematics required for action calculations; guidance is given in ISO 19901-1:2005, 5.2 and 8.3. Unless site-specific information indicates otherwise, T_{ass} is normally between the limits as given in Equation (A.6.4-8):

$$3,44\sqrt{(H_{\text{srp}})} < T_{\text{ass}} < 4,42\sqrt{(H_{\text{srp}})} \quad (\text{A.6.4-8})$$

where H_{srp} is the return period of the extreme significant wave height, expressed in metres.

A.6.4.2.4 Wave crest elevation

The wave crest elevation used to determine the minimum hull elevation above the extreme still water level in A.6.4.4 can be obtained from the extreme wave height, H_{max} in A.6.4.2.2, and the appropriate deterministic wave theory in A.7.3.3.3.1.

A reasonably foreseeable extreme return period should be used for this calculation, and should be no shorter than 50 years, even if a lower return period is used for other purposes (e.g. the ULS assessment in tropical storm areas).

For some regions, the abnormal wave crest elevation should be calculated based on storm statistics and according to principles described in ISO 19901-1:2005, A.8.8. Examples for the regional application of these principles can 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 account for wave spreading and the conservatism of deterministic (regular) wave kinematics (see A.6.4.2.3), it should not be applied in the calculation of the wave crest elevation.

A.6.4.2.5 Wave spectrum

Where the analysis method requires the use of spectral data, the choice of the analytical wave spectrum and associated spectral parameters should reflect the width and shape of the spectra for the site and the significant wave height under consideration. In cases where the fetch and duration of extreme winds are sufficiently long, a fully developed sea results (this is rarely realized except, for example, in areas subject to monsoons). Such conditions can be represented by a Pierson-Moskowitz spectrum. Where the fetch or duration of extreme winds is limited, or in shallow water depths, a JONSWAP spectrum can normally be applied (see A.6.4.2.7).

Further discussions of wave spectra and spectral density functions for the Pierson-Moskowitz, $S_{\text{PM}}(\omega)$, and the JONSWAP, $S_{\text{JS}}(\omega)$, wave spectra are presented in ISO 19901-1:2005 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 19905-2:2012, 6.4.2.5.

A.6.4.2.6 Airy wave height correction for stochastic analysis

When Airy wave theory is used for stochastic (random) wave action calculations, see A.7.3.3.3.2, then it is necessary to account for wave asymmetry, which is not included in Airy wave theory. The significant wave height should be increased to capture the largest wave actions at the maximum crest amplitude. The increased significant wave height, H_s , should be determined as a function of the water depth, d , expressed in metres, as given in Equation (A.6.4-9):

$$H_s = [1 + (10H_{srp}/T_{p,i}^2) e^{(-d/25)}] H_{srp} \tag{A.6.4-9}$$

where

- d is the still, or undisturbed, water depth (positive);
- H_{srp} is the return period extreme significant wave height, expressed in metres;
- $T_{p,i}$ is the intrinsic modal or peak period of the wave spectrum, and should be used with the wave kinematics model described in A.7.3.3.3.2.

A.6.4.2.7 Peak and zero-upcrossing periods

When undertaking a stochastic analysis (either for a one-stage analysis or for determining a DAF for a two-stage analysis), it is necessary to either consider a range of wave periods or a suitable wave spectrum that contains sufficient breadth of the peak to capture the dynamic characteristics. Information on the range of periods to use is given in A.6.4.2.7, however, to avoid the requirement for dynamic analyses with several different wave periods, a practical alternative is to use a two-parameter spectrum, such as Pierson-Moskowitz with $\gamma = 1,0$, in combination with the site-specific most probable peak period; when using the relationships in Table A.6.4-1, the value of γ used should be as given by the data provider.

For a given significant wave height the wave period depends on the significant wave steepness which in extreme seas in deep water often lies within the range 1/20 to 1/16. This leads to the expression for intrinsic zero-upcrossing period $T_{z,i}$, related to H_{srp} in metres, given in Equation (A.6.4-10):

$$3,2\sqrt{(H_{srp})} < T_{z,i} < 3,6\sqrt{(H_{srp})} \tag{A.6.4-10}$$

However in shallow water the wave steepness can increase to 1/12 or more, leading to an intrinsic zero-upcrossing period $T_{z,i}$ as low as $2,8\sqrt{(H_{srp})}$. This is because in shallow water the wave height increases and wave length decreases for a given $T_{z,i}$.

When considering a JONSWAP spectrum, the peak enhancement factor γ varies between 1 and 7 with a most probable average value of 3,3. There is no firm relationship between γ , H_s and $T_{p,i}$. Relationships between variables for different γ according to Reference [A.6.4-4] are given in Table A.6.4-1.

Table A.6.4-1 — Relationship between γ , H_s and $T_{p,i}$

γ	$T_{p,i}/T_{z,i}$
1	1,406
2	1,339
3	1,295
3,3	1,286
4	1,260
5	1,241
6	1,221
7	1,205

Unless site-specific information indicates otherwise $\gamma = 3,3$ can be used.

If a JONSWAP spectrum is applied, the response analysis should consider a range of periods associated with H_{srp} based on the most probable value of $T_{p,i}$ plus or minus one standard deviation. However, it should be ensured that the assumptions made in deriving the spectral period parameters are consistent with the values used in the analysis. Alternatively, applicable combinations of wave height and period can be obtained from a scatter diagram determined from site-specific measurements; in this case, specialist advice should be obtained on a suitable spectral form for the site.

For other spectrums the assessor is referred to DNV-RP-C205^[A.6.4-3] for guidance.

A.6.4.2.8 Short-crested stochastic waves

For calculations of stochastic (random) wave actions, the short-crestedness of waves (i.e. the angular distribution of wave energy about the dominant direction) may be taken into account when site-specific information indicates that such effects are applicable. In all cases the potential for increased response due to short-crested waves should be investigated. The effect may be included by means of a directionality function $F(\alpha)$, given in Equation (A.6.4-11):

$$S_{\eta\eta}(f, \alpha) = S_{\eta\eta}(f) \cdot F(\alpha) \quad (\text{A.6.4-11})$$

where

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

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

$F(\alpha)$ is the directionality function.

Directionality functions for extreme and fatigue analyses can be found in ISO 19901-1:2005, A.8.7, and ISO/TR 19905-2:2012, 6.4.2.8. When referring to the formulations in ISO 19901-1:2005, 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 19901-1:2005, A.8.7, then the directional spreading function D_1 with $n = 8$ gives good agreement with the formulation in ISO/TR 19905-2:2012, 6.4.2.8. For directional spreading function D_2 with $s = 15$ and for directional spreading function D_3 with $\sigma = 0,34$ there is good agreement with the formulation in ISO/TR 19905-2:2012, 6.4.2.8.

The modelling of short-crested stochastic waves should not be combined with the wave kinematics factor used in deterministic wave analysis to represent wave spreading and the conservatism of deterministic (regular) wave kinematics; see A.6.4.2.3.

A.6.4.2.9 Maximizing the wave/current response

Where the natural period of the jack-up is such that it can respond dynamically to waves; see A.10.4.1, the maximum dynamic response can be caused by waves or sea states with periods outside the ranges given in A.6.4.2.3 and A.6.4.2.7. Such conditions should also be investigated to ensure that the maximum (dynamic plus quasi-static) response is determined by considering sea states with different combinations of significant wave height and spectral period, or deterministic waves with different combinations of individual wave height and period. Such combinations may be limited to probabilities of exceedance that are equal to or lower than the intended probability level of the assessment.

A.6.4.2.10 Long-term wave data

For fatigue calculations (see 11.3.1), the long-term wave climate is required. For fatigue analysis, the long-term data present the probability of occurrence for each sea state, characterized by wave energy spectra and the associated physical parameters. This can be presented in the form of a significant wave height versus zero-upcrossing period scatter diagram or as a table of representative sea states.

A.6.4.3 Current

Current components should be applied in the downwind direction. The extreme wind driven surface current velocity should be that associated with the assessment return period wind. When directional information regarding other current velocity components is available, the downwind component of the maximum surface flow of the mean spring tidal current and the assessment return period surge current should be added to the wind driven surface current as indicated below. When appropriate, the currents can be seasonally adjusted. If directional data are not available, the components should be summed algebraically and assumed to be omnidirectional.

A site-specific study should normally define the current velocity components.

The current profile can be defined by a series of velocities at a range of elevations from sea floor to water surface. Unless site-specific data indicates otherwise, and in the absence of other residual currents (such as circulation, eddy currents, slope currents, internal waves, inertial currents, etc.), an appropriate method for computing current profile (see Figure A.6.4-1) is as given in Equations (A.6.4-12) and (A.6.4-13):

$$V_C = V_t + V_s + (V_w - V_s) [(h + z)/h] \quad \text{for } |z| \leq h \text{ and } V_s < V_w \quad (\text{A.6.4-12})$$

$$V_C = V_t + V_s \quad \text{for } |z| > h \text{ or } V_s \leq V_w \quad (\text{A.6.4-13})$$

where

V_C is the current velocity as a function of z ;

NOTE A reduction can be applicable according to A.7.3.3.4.

V_t is the downwind component of mean spring tidal current;

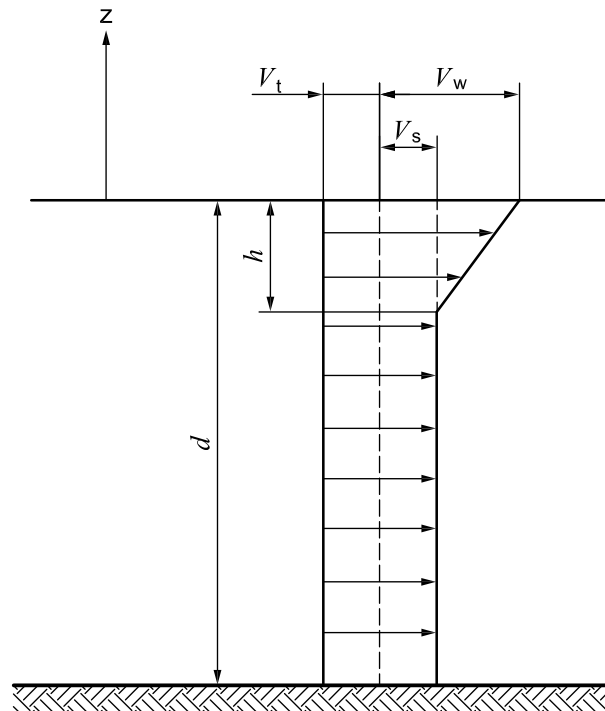
V_s is the downwind component of associated surge current (excluding wind driven component);

V_w is the wind generated surface current; in the absence of other data, this may conservatively be taken as 2,6 % of the 1 min sustained wind velocity at 10 m;

h is the reference depth for wind driven current, in the absence of other data, h should be taken as 10 m;

z is the vertical coordinate relative to the SWL under consideration, positive upwards (always negative in the water column).

Alternative formulations are provided in ISO 19901-1:2005, A.9.3. Comparisons of combined current and wave actions in ISO/TR 19905-2:2012, 6.4.3, show that the constant current profile is on the conservative side compared to the power law formulations presented in ISO 19901-1.



Key

- d water depth
- h reference depth for wind-driven current
- V_s downwind component of surge current
- V_t downwind component of tidal current
- V_w wind driven surface current
- z vertical coordinate relative to the SWL under consideration, positive upwards

Figure A.6.4-1 — Suggested current profile

In the presence of waves the current profile should be stretched/compressed such that the surface component remains constant. This can be achieved by substituting the elevation as described in A.7.3.3.3.2. Alternative methods can be suitable, however mass continuity methods are not recommended.

The current profile can be changed by wave breaking. In such cases the wind induced current could be more uniform with depth.

For a fatigue analysis, current can normally be neglected.

A.6.4.4 Water depths

The mean sea level (MSL) related to the sea floor is defined in 3.40.

The SWLs used for the assessment of the site should be determined and related to LAT. The relationship between LAT and CD is discussed in ISO/TR 19905-2:2012, 6.4.4.

- Different extreme water levels are required for the ULS assessment and hull elevation determination:
 - Unless reliable joint probability data are available, the extreme SWL, expressed as a height above LAT can be taken as follows:
 - mean high water spring tide + relevant return period extreme storm surge.

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

mean low water spring tide + relevant return period negative storm surge.

- When determining the SWL for air gap calculations (safe hull elevation), a reasonably foreseeable extreme return period should be used. This should be no shorter than 50 years, even if a lower return period is used for other purposes (e.g. the ULS assessment in tropical storm areas).

A.6.4.5 Marine growth

Site-specific data should be obtained. In the absence of such data, default values for thickness and distribution are given in A.7.3.2.5.

A.6.4.6 Wind

A.6.4.6.1 General

The wind velocity used for the assessment return period should be the 1 min sustained wind, related to a reference level of 10 m above MSL.

The wind velocity profile may be defined by a logarithmic function according to ISO 19901-1, or approximated by a power law (see A.6.4.5.2). A comparison of wind actions shows that the power law profile is slightly more severe than the ISO 19901-1 logarithmic profile, see ISO/TR 19905-2:2012,6.4.5.1. Typically, the average difference is in the range of 7 % for a 1 min average wind speed of 20 m/s at 10 m above sea level, and 2 % for a 1 min average wind speed of 40 m/s.

Different jack-up configurations (weight, centre of gravity, cantilever position, etc.) may be specified for operating and elevated storm modes. In such cases, the maximum wind velocity considered for the operating mode should not exceed that permitted for the change to the elevated storm mode.

Equations for the 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 given by Equations (A.6.4-14) and (A.6.4-15):

$$V_Z = V_{\text{ref}} (Z/Z_{\text{ref}})^{1/N} \quad \text{for } Z \geq Z_{\text{ref}} \quad (\text{A.6.4-14})$$

$$V_Z = V_{\text{ref}} \quad \text{for } Z < Z_{\text{ref}} \quad (\text{A.6.4-15})$$

where

V_Z is the wind velocity at elevation Z above the SWL under consideration;

V_{ref} is the 1 min sustained wind velocity at elevation Z_{ref} (normally 10 m);

Z is the elevation above the SWL;

Z_{ref} is the reference elevation above the SWL;

N is the inverse exponent of the power law profile; $N = 10$ 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 site and the foundation stability. The area covered should be sufficiently large to encompass any stand-off location; normally a 1 km × 1 km square is sufficient. Aspects that should be investigated are shown in Table A.6.5-1 and are discussed in more detail in the referenced subclauses. The information obtained from the surveys and investigations set out in A.6.5.1.2 to A.6.5.1.5 is required for areas where there is no adequate data available from previous operations. In areas where information is available, the recommendations set out herein may be considered using information obtained from other surveys or activities in the field.

Experience of prior jack-up operations in the same field may be used provided that the previous bearing pressures exceed those for the present operation by an adequate margin.

A.6.5.1.2 Bathymetric survey

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

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) around the intended site. The slant range selection should give a minimum of 100 % overlap between adjacent lines. A magnetometer survey should also be undertaken if there are buried pipelines, cables and other metallic debris located on or slightly below the sea floor.

Sufficient information should be obtained to enable safe positioning and removal of the jack-up. Sea floor obstructions, such as pipelines and wellheads, should be identified to sufficient depth to avoid the potential for spudcan interference during both installation on and removal from site. In some cases 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.

Table A.6.5-1 — Foundation risks, methods for evaluation and prevention

Risk	Methods for evaluation and prevention	Subclause
Installation problems	Bathymetric survey	A.6.5.1.2
	Sea floor survey	A.6.5.1.3
Punch-through	Shallow seismic survey	A.6.5.1.4
	Soil sampling and other geotechnical testing and analysis	A.6.5.1.5, A.9.3.6
Settlement/bearing failure	Shallow seismic survey	A.6.5.1.4
	Soil sampling and other geotechnical testing and analysis	A.6.5.1.5, A.9.3.6
	Ensure adequate jack-up preload capability	A.9.3.6
Sliding failure	Shallow seismic survey	A.6.5.1.4
	Soil sampling and other geotechnical testing and analysis	A.6.5.1.5, A.9.3.6
	Increase vertical spudcan reaction	
	Modify the spudcans	
Scour	Bathymetric and sea floor survey (identify sand waves)	A.6.5.1.2
	Surface soil samples and sea floor currents	A.6.5.1.3
	Inspect spudcan foundation regularly	
	Install scour protection (gravel bag/artificial seaweed) when anticipated	A.9.4.7
Geohazards (mudslides, mud volcanoes etc)	Sea floor survey	A.6.5.1.3
	Shallow seismic survey	A.6.5.1.4
	Soil sampling and other geotechnical testing and analysis	A.6.5.1.5
Gas pockets/shallow gas	Shallow seismic survey	A.6.5.1.4
Faults	Shallow seismic survey	A.6.5.1.4
Metal or other object, sunken wreck, anchors, pipelines etc.	Magnetometer and sea floor survey	A.6.5.1.3
Local holes (depressions) in sea floor, reefs, pinnacle rocks, non-metallic structures or wooden wreck	Sea floor survey	A.6.5.1.3
	Diver/ROV inspection	
Leg extraction difficulties	Soil sampling and other geotechnical testing and analysis	A.6.5.1.5, A.9.4.5
	Consider change in spudcans	
	Jetting/Airlifting	A.9.4.5
Eccentric spudcan reactions	Bathymetry, sea floor & shallow seismic surveys	A.6.5.1.2, A.6.5.1.3, A.6.5.1.4
	Shallow seismic survey (buried channels or footprints)	A.6.5.1.4
	Soil sampling and other geotechnical testing and analysis	A.6.5.1.5, A.9.4.2
	Seabed modification	
Seabed slope	Bathymetry, sea floor & shallow seismic survey	A.6.5.1.2, A.6.5.1.3, A.6.5.1.4
	Seabed modification	A.9.4.2
Footprints of previous jack-ups	Evaluate field records	A.6.5.1.1, A.6.5.1.2, A.6.5.1.3
	Prescribed installation procedures	A.9.4.3
	Consider filling/modification of holes as necessary	A.9.4.3

A.6.5.1.4 Shallow seismic survey

A shallow seismic survey uses high resolution acoustic reflection techniques to

- determine near surface soil stratigraphy;
- 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 km square area centred on the proposed site. Line spacing of the survey should typically be not greater than 100 m × 250 m over the survey area. The survey report should include at least two vertical cross-sections passing through the proposed site showing all the relevant reflectors and allied geological information. The equipment used should be capable of identifying reflectors of 0,5 m and thicker to a depth equal to the greater of 30 m or the anticipated spudcan penetration plus 1,5 times the spudcan diameter.

A.6.5.1.5 Geotechnical investigation

A.6.5.1.5.1 General

Site-specific geotechnical investigation and testing are recommended in areas where any of the following apply.

- Nearby geotechnical data are not available.
- The shallow seismic survey cannot be interpreted with any certainty.
- Significant layering of the strata is indicated.
- The site is known to be potentially hazardous.

A.6.5.1.5.2 Soil investigation and testing

A geotechnical investigation should comprise a minimum of one borehole to a depth below the sea floor of 30 m or the anticipated spudcan penetration plus 1,5 times the spudcan diameter, whichever is the greater. All the layers should be adequately investigated and the transition zones cored at a sufficient sampling rate.

The number of boreholes should account for the lateral variability of the soil conditions, regional experience and the geophysical investigation. When a single borehole is made, the borehole should be at the centre of the leg pattern. More detailed recommendations from the InSafeJIP^[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 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, submerged unit weight and the over consolidation ratio (R_{OC}).

Additional soil testing to provide shear moduli for cyclic or dynamic behaviour should be undertaken if more comprehensive analyses are needed 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 site. Lateral variations of geotechnical parameters can be assessed from the correlation of the shallow seismic data and the geotechnical information from the borehole logs and/or *in-situ* tests.

A.6.6 Earthquake data

No guidance is offered.

A.7 Actions

A.7.1 Applicability

Clause A.7 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 in A.7.3. Normally a quasi-static, deterministic extreme wave analysis is performed for jack-up site-specific assessments, and the dynamic effects are represented by an inertial loadset. Calculations of actions for stochastic analysis in time domain simulations are also presented. Such analyses are applicable for calculation of inertial loadsets or for the direct calculation of the structural responses including dynamic effects. The hydrodynamic formulations and coefficients are presented together with equations for detailed and equivalent modelling of leg hydrodynamic actions.

Wind models, flow coefficients for different structural parts and a formulation for the calculation of static wind actions are presented in A.7.3.4.

Guidance on the determination of the functional actions is presented in A.7.4.

A.7.2 General

No guidance is offered.

A.7.3 Metocean actions

A.7.3.1 General

A.7.3.1.1 Load cases

The wave/current actions on the legs and other structures and the wind actions on the hull, legs and other structures should be considered due to either

- a) the 50 year return period individual extremes, or
- b) the most onerous combinations of the following 100 year joint probability metocean data:
 - 1) 100 year return period wave, the associated current and associated wind;
 - 2) 100 year 1 min wind, the associated wave and associated current;
 - 3) 100 year current and the associated wave and associated wind.

A.7.3.1.2 Methods for the determination of actions

This subclause describes how the actions are developed for determining the jack-up response by one of two alternative methods, deterministic and stochastic.

A deterministic analysis involves developing static metocean actions and an inertial loadset. The inertial loadset can be developed from either an SDOF method or a stochastic assessment of the wave actions to develop a DAF.

A more detailed stochastic time domain analysis procedure implicitly includes inertial actions and can account for non-linearities of the action and foundation interaction.

The action calculation procedure should follow the steps in the applicable column of Table A.7.3-1.

Table A.7.3-1 — Metocean action calculation procedures

Topic	Description	Deterministic analysis	Stochastic DAF method	Fully integrated stochastic analysis
Water depth	Define storm water depth considering LAT, tide and storm surge	A.6.4.4		
Current	Define current velocity and profile.	A.6.4.3		
	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.	A.7.3.3.4		
	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.	A.6.4.3		
Wave	Specify wave height and range of associated wave periods.	A.6.4.2.2 A.6.4.2.3		
	Determine if supplied wave periods are intrinsic or apparent and calculate the other value that has not been supplied	A.7.3.3.5, ISO 19901-1:2005, 8.3 and A.8.3		
	Define the return period significant wave height and corresponding spectral peak period	not applicable	A.6.4.2.5, A.6.4.2.7	
	Calculate effective significant wave height as appropriate	not applicable	A.6.4.2.6	
	Specify wave spectrum, wave direction and wave spreading function	not applicable	A.6.4.2.5, A.6.4.2.8	
	Calculate wave velocities and accelerations by superposition of intrinsic wave components representing the wave spectrum and wave spreading functions	not applicable	A.7.3.3.3.2	
	Is deterministic wave subject to cancellation?	A.10.4.2.5		not applicable
Wave theory	Determine the two-dimensional wave kinematics from an appropriate wave theory for the specified wave height, storm water depth, and intrinsic wave period	A.7.3.3.3.1	not applicable	
	Apply a reduction factor to the wave kinematics	A.6.4.2.3	not applicable	
Scale the environment	Apply partial factors to wind, wave and current to match factored deterministic actions	not applicable		A.10.5.3.2

Table A.7.3-1 (continued)

Topic	Description	Deterministic analysis	Stochastic DAF method	Fully integrated stochastic analysis
Hydrodynamic modelling	Establish detailed or equivalent leg models to represent structural members and appurtenances	A.7.3.2.1, A.7.3.2.2, A.7.3.2.3, A.7.3.2.6		
	Determine drag and inertia coefficients (detailed or equivalent) as functions of member shape, roughness (marine growth), size, and orientation.	A.7.3.2.4, A.7.3.2.5		
	Include the marine growth thickness relevant for the site and duration of the planned operation	A.7.3.2.5		
Wave/current action	Combine local current profile vectorially with the wave kinematics to determine locally incident fluid velocities and accelerations for calculation of wave and current actions by Morison's equation.	A.7.3.3.3.1, A.7.3.3.3.2		
Wind	Define wind speed and wind profile	A.6.4.6		
Wind action	Define shape coefficients and calculate the static wind action.	A.7.3.4		
Functional actions	Define functional actions	A.7.4		
Other actions	Define other actions	A.7.8		
Stochastic DAF	Does natural period coincide with cancellation or reinforcement	not applicable	A.7.3.3.3.3, A.10.4.2.5	not applicable
	Determine DAF stochastically	not applicable	A.10.5.2.2.3, A.10.5.3	not applicable
Dynamic effects	Determine DAF either deterministically or stochastically. Represent dynamic effects by an inertial loadset	A.10.5.2.2.2 A.10.5.2.2.3	← follow deterministic analysis	not applicable
	Does natural period coincide with cancellation or reinforcement?	not applicable	not applicable	A.7.3.3.3.3, A.10.4.2.5
Action factors	Apply action factors to the metocean actions and dynamic effects	8.8.1.2	not applicable	8.8.1.3
Load cases	Develop assessment load case by linearly combining the factored metocean actions with the factored functional actions	8.8.1.1, A.10.5.2.2.3	not applicable	8.8.1.1
	Additional load cases if $(T_n/T_p) > 0,9$	A.10.5.2.2.3	not applicable	not applicable

When a fully integrated stochastic analysis is undertaken (see 10.3), partial factors are applied to the metocean parameters instead of the metocean actions, as described in A.10.5.3 and 8.8.1.3. When using stochastic dynamic analyses for the purpose of determining a DAF, no partial action factors are applied; however, in the subsequent deterministic analysis including the inertial loadset based on the stochastic DAF, the action factors described in 8.8.1.2 are applied.

A.7.3.2 Hydrodynamic model

A.7.3.2.1 General

The hydrodynamic modelling of the jack-up leg can be carried out by utilizing “detailed” or “equivalent” techniques. The hydrodynamic properties are then found as described in A.7.3.2.2 to A.7.3.2.4. In all cases, the provisions in the remainder of A.7.3.2.1 should be considered.

The drag properties of some chords represented by the product of the drag coefficient C_D and diameter D differ for flow in the direction of the wave propagation (in the wave crest) and for flow back in the opposite direction (in the wave trough). Often the combined drag properties of all the chords on a leg gives a total value along a particular axis that is independent of the flow direction. When this is not the case, it is recommended that the effect is included directly in the wave/current action model. Otherwise, where possible it is recommended that

- a) regular wave deterministic calculations use drag properties appropriate to the flow direction under consideration, noting that the flow direction is that of the combined wave particle motion and current;
- b) for random wave analyses which are solely used to determine dynamic effects for inclusion in a final regular wave deterministic calculation on the basis of item a) above, an average drag property is considered;
- c) for random wave analyses from which the final results are obtained directly the drag property in the direction of wave propagation is used.

Lengths of members are normally taken as the node-to-node distance of the members in order to account for small non-structural items (e.g. anodes, jetting lines of less than 4" nominal diameter); see note below. Large non-structural items, such as raw water pipes and ladders, should be included in the model. Free standing conductor pipes and raw water towers should be considered separately from the leg hydrodynamic model.

NOTE 1 For the purpose of this calculation, a node is defined as the point where two member axes intersect. Offsets between terminating members along the axis of the continuous member at the node may be used when calculating the equivalent C_D .

The contribution of the part of the spudcan above the sea floor should be investigated and only excluded from the model if it is shown to be insignificant. In water depths greater than $2,5H_s$ or where penetrations exceed half the spudcan height, the effect of the spudcan is normally insignificant. Otherwise, hydrodynamic actions should be modelled with hydrodynamic coefficients applicable for large diameter members; see ISO/TR 19905-2:2012, 7.3.2.4 and 7.3.2.5.

On some jack-ups, the lower section of the leg adjacent to the spudcan can be heavily reinforced for towage; this should be explicitly modelled.

For leg structural members, shielding and solidification effects should not normally be applied in calculating wave actions. The current flow is however reduced due to interference from the structure on the flow field, see A.7.3.3.4.

NOTE 2 The solidification effect, which increases the actions from waves due to interference from objects “side by side” in the flow field, is normally not included in the determination of the hydrodynamic coefficients or jack-ups. Jack-ups are usually space frame structures with few parallel members in close proximity so that shielding and solidification effects are usually not important. However, solidification can be important for closely spaced members such as are found in some raw water systems.

Coefficients for individual members with closely attached appurtenances should be calculated by accounting for the combined shape with reference to relevant literature^[A.7.3-1]. Model test data may be used for non-circular members, if available. In such cases the effects of roughness, Keulegan-Carpenter and Reynolds number dependence should be considered. The building block methodology described below was developed and calibrated for SNAME Technical and Research Bulletin 5-5A^[7]. Model tests and analytical studies for complete legs are difficult to interpret and are unlikely to give results that are consistent with the methodology used here. This is particularly true for legs in which tubular members contribute a significantly to the total drag coefficient because of Reynolds number dependency.

A.7.3.2.2 “Detailed” leg model

All members are modelled with Morison coefficients accounting for member cross-section orientation relative to the flow direction. Members can be lumped together using the corresponding $C_D D = \sum C_{Di} D_i$ and $C_M A = \sum C_{Mi} \pi D_i^2 / 4$, accounting for flow direction, as defined in A.7.3.2.4.

A.7.3.2.3 “Equivalent” leg model

The hydrodynamic model of a bay is comprised of one, “equivalent” vertical tubular located at the geometric centre of the actual leg. The corresponding (horizontal) v_n , u_n and i_n are applied together with equivalent $C_D D = \sum C_{De} D_e$ and $C_M A = \sum C_{Me} A_e$, as defined in A.7.3.2.4. The model should be varied with elevation, as necessary, to account for changes in dimensions, marine growth thickness, etc.

When the hydrodynamic properties of a lattice leg are idealized by an “equivalent” model, the properties can be found using the method given below.

The equivalent value of the drag coefficient, C_{De} , times the equivalent diameter, D_e , of the bay can be chosen as given in Equation (A.7.3-1):

$$C_{De} D_e = D_e \sum C_{Dei} \tag{A.7.3-1}$$

The equivalent value of the drag coefficient for each member, C_{Dei} , is determined as given in Equation (A.7.3 2):

$$C_{Dei} = [\sin^2 \beta_i + \cos^2 \beta_i \sin^2 \alpha_i]^{3/2} C_{Di} \frac{D_i l_i}{D_e s} \tag{A.7.3-2}$$

where

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

NOTE 1 Σ indicates summation over all members in one leg bay.

The above expression for C_{Dei} can be simplified for horizontal and vertical members as given in Equations (A.7.3-3) and (A.7.3-4):

vertical members (e.g. chords): $C_{Dei} = C_{Di} (D_i / D_e)$ (A.7.3-3)

horizontal members: $C_{Dei} = \sin^3 \alpha_i C_{Di} \left(\frac{D_i l_i}{D_e s} \right)$ (A.7.3-4)

The equivalent value of the inertia coefficient, C_{Me} , and the equivalent area, A_e , representing the bay can be determined from the following:

C_{Me} is the equivalent inertia coefficient, which may normally be taken as 2,0 when using A_e .

A_e , the equivalent area of leg per unit height, is equal to $(\sum A_i l_i)/s$.

A_i , the equivalent area of member or gusset, is equal to $\pi D_i^2/4$.

D_i , the reference diameter, is chosen as defined in A.7.3.2.4.

For a more accurate model, the C_{Me} coefficient may be determined as given in Equation (A.7.3-5):

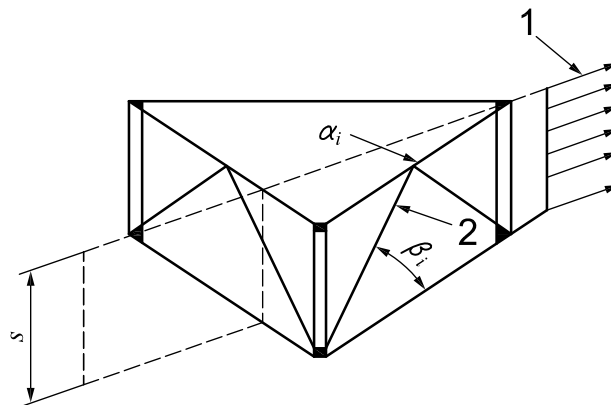
$$C_{Me} A_e = A_e \sum C_{Mei} \quad (\text{A.7.3-5})$$

where

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

C_{Mi} is the inertia coefficient of an individual member, which is defined in A.7.3.2.4 related to reference dimension D_i .

NOTE 2 For dynamic modelling the added mass of fluid per unit height of leg may be determined as $\rho A_i (C_{Mi} - 1)$ for a single member or $\rho A_e (C_{Me} - 1)$ for the equivalent model, provided that A_e is as defined above.



Key

1 flow direction

2 member i

s bay height

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

β_i angle defining the inclination of member i from horizontal

NOTE Based on DNV Class Note 31.5, February 1992^[A.7.3-2].

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

A.7.3.2.4 Drag and inertia coefficients

Hydrodynamic coefficients for leg members are given in this subclause. Tubulars, brackets, split tube and triangular chords are considered. Hydrodynamic coefficients including directional dependence are given together with a fixed reference diameter D_i . No other diameter should be used unless the coefficients are scaled accordingly. Unless better information is available for the computation of wave/current actions, the values of drag and inertia coefficients applicable to Morison's equation should be obtained from this subclause.

Recommended values for hydrodynamic coefficients for tubulars with a diameter smaller than 1,5 m are given in Table A.7.3-2, based on the data discussed in the supporting ISO/TR 19905-2:2012, 7.3.2.4.

Table A.7.3-2 — Base hydrodynamic coefficients for tubulars

Surface condition	C_{Di}	C_{Mi} for wave load analysis	C_{Mi} for earthquake
Smooth	0,65	2,0	2,0
Rough	1,00	1,8	2,0

The smooth values normally apply above MSL + 2 m and the rough values below MSL + 2 m, where MSL is as defined in A.6.4.4. If the jack-up has operated in deeper water and the fouled legs are not cleaned the surface should be taken as rough for wave actions above MSL + 2 m.

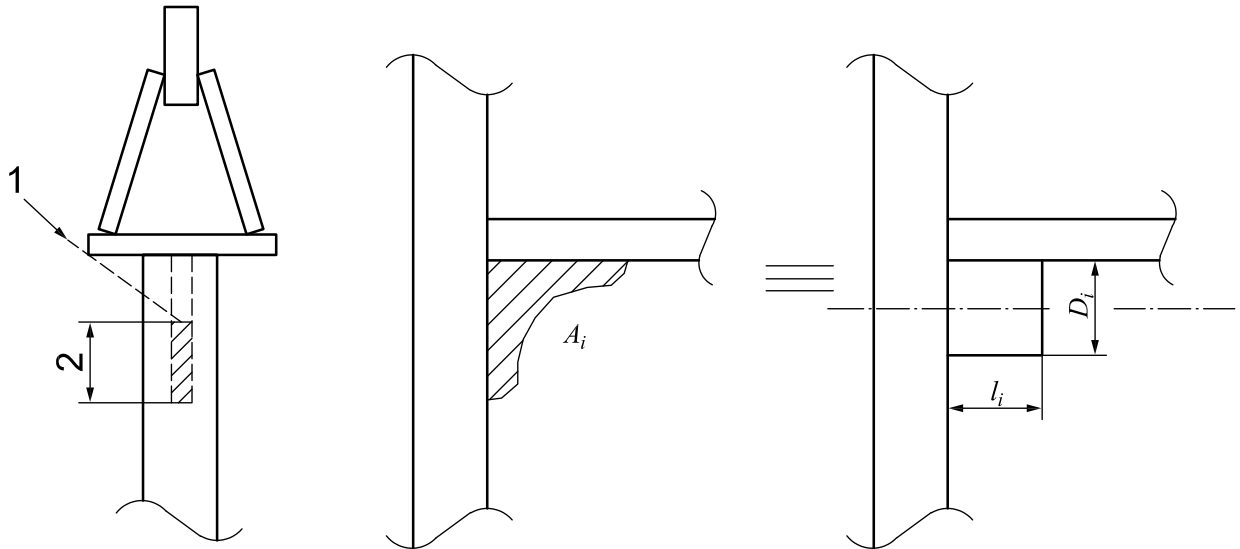
Hydrodynamic coefficients for large diameter members may be calculated according to ISO/TR 19905-2:2012, 7.3.2.4 and 7.3.2.5.

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

$$C_{Di} = 2,0$$

applied together with the projected area of the gusset visible in the flow direction, unless model test data show otherwise. This drag coefficient may be applied together with a reference diameter D_i and corresponding length l_i chosen such that their product equals the plane area, $A_i = D_i l_i$ and $D_i = l_i$ (see Figure A.7.3-2). In the equivalent model of A.7.3.2.3 the gussets may be treated as an equivalent horizontal member of length l_i , with its axis in the plane of the gusset. C_{Mi} should be taken as 1,0 and marine growth may be ignored.

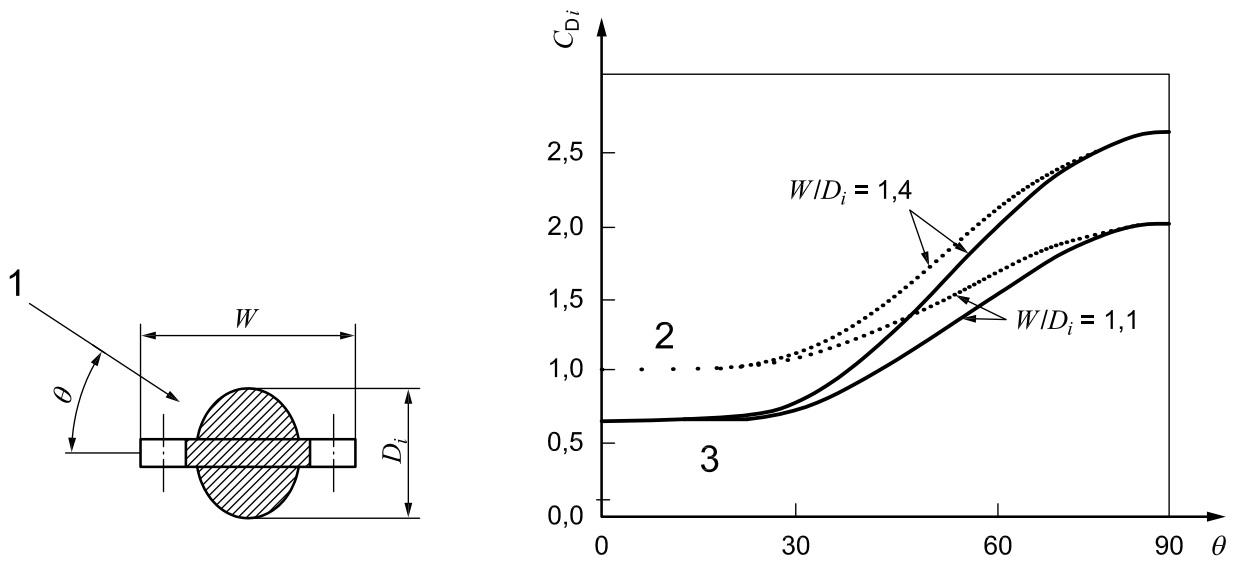
For non-tubular geometries (e.g. leg chords) the appropriate hydrodynamic coefficients may, in lieu of more detailed information, be taken in accordance with Figure A.7.3-3 or Figure A.7.3-4 and corresponding equations, as appropriate.



Key

- 1 flow direction
- 2 visible part of gusset *i*
- A_i area of gusset *i*; $A_i = l_i D_i$
- D_i reference diameter of gusset *i*
- l_i reference length of gusset *i*

Figure A.7.3-2 — Gusset plates: equivalent modelling



Key

- 1 flow direction
- 2 rough
- 3 smooth
- C_{Di} drag coefficient for use with D_i
- D_i reference dimension of chord *i*
- W average width of the rack
- θ angle between flow direction and plane of rack (degrees)

Figure A.7.3-3 — Split tube chord and typical values for C_{Di}

For a split tube chord as shown in Figure A.7.3-3 the drag coefficient C_{Di} , related to the reference dimension $D_i = D + 2t_m$, the diameter of the tubular, including marine growth as in A.7.3.2.3, should be taken from Equation (A.7.3-7):

$$C_{Di} = \begin{cases} C_{Do} & ; \quad 0^\circ < \theta \leq 20^\circ \\ C_{Do} + \left(C_{D1} \frac{W}{D_i} - C_{Do} \right) \sin^2 [(\theta - 20^\circ) 9 / 7] & ; \quad 20^\circ < \theta \leq 90^\circ \end{cases} \quad (A.7.3-7)$$

where

t_m is the marine growth thickness;

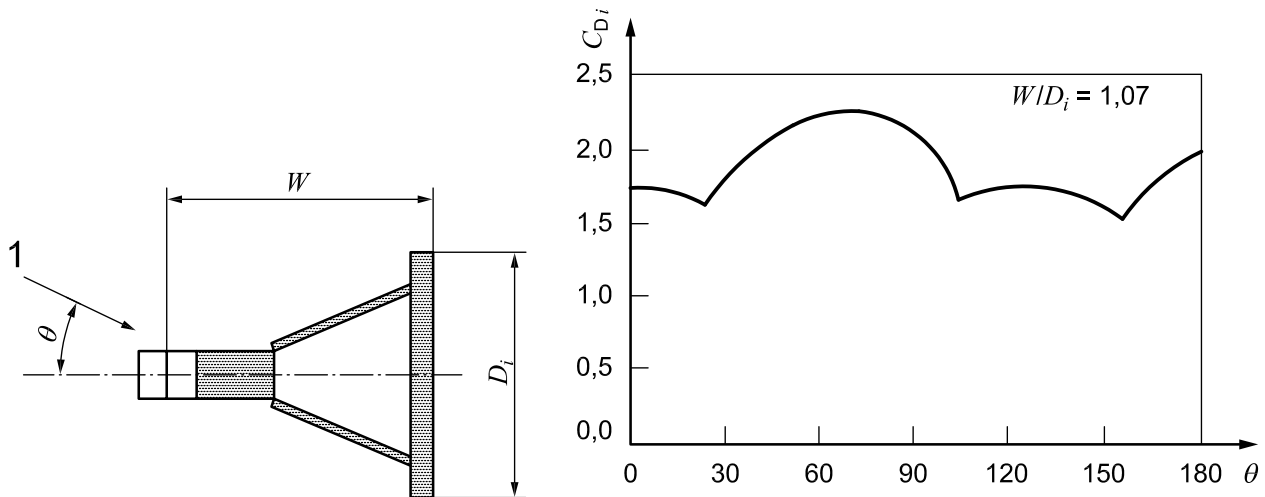
θ is the angle in degrees; see Figure A.7.3-3;

C_{Do} is the drag coefficient for a tubular with appropriate roughness, see Table A.7.3-2;

C_{D1} is the drag coefficient for flow normal to the rack ($\theta = 90^\circ$), related to projected diameter, W . C_{D1} is given by Equation (A.7.3-8):

$$C_{D1} = \begin{cases} 1,8 & ; \quad W / D_i < 1,2 \\ 1,4 + 1/3(W / D_i) & ; \quad 1,2 < W / D_i < 1,8 \\ 2,0 & ; \quad 1,8 < W / D_i \end{cases} \quad (A.7.3-8)$$

The inertia coefficient $C_{Mi} = 2,0$, related to the equivalent volume $\pi D_i^2 / 4$ per unit length of member, can be applied to all heading angles and any roughness.



Key

- 1 flow direction
- C_{Di} drag coefficient for use with D_i
- D_i reference dimension (height of backplate) of chord i
- W width of chord to mid-point of rack tooth
- θ angle between flow direction and plane of rack (degrees)

Figure A.7.3-4 — Triangular chord and typical values of C_{Di}

For a triangular chord as shown in Figure A.7.3-4, the drag coefficient C_{Di} related to the reference dimension $D_i = D$, the backplate width, should be taken from Equation (A.7.3-9):

$$C_{Di} = C_{Dpr}(\theta) D_{pr}(\theta) / D_i \quad (\text{A.7.3-9})$$

where the drag coefficient related to the projected diameter, C_{Dpr} , is determined from Equation (A.7.3-10):

$$C_{Dpr}(\theta) = \begin{cases} 1,70 & ; & \theta = 0^\circ \\ 1,95 & ; & \theta = 90^\circ \\ 1,40 & ; & \theta = 105^\circ \\ 1,65 & ; & \theta = 180^\circ - \theta_0 \\ 2,00 & ; & \theta = 180^\circ \end{cases} \quad (\text{A.7.3-10})$$

Linear interpolation should be applied for intermediate headings. The projected diameter, $D_{pr}(\theta)$, should be determined from Equation (A.7.3-11):

$$D_{pr}(\theta) = \begin{cases} D_i \cos \theta & ; & 0 < \theta < \theta_0 \\ W \sin \theta + 0,5 D_i |\cos \theta| & ; & \theta_0 < \theta < 180 - \theta_0 \\ D_i |\cos \theta| & ; & 180 - \theta_0 < \theta < 180 \end{cases} \quad (\text{A.7.3-11})$$

The angle θ_0 is the angle where half the rackplate is hidden, $\theta_0 = \tan^{-1}[D_i/(2W)]$.

The inertia coefficient $C_{Mi} = 2,0$ (as for a flat plate), related to the equivalent volume of $\pi D_i^2/4$ per unit length of member, can be applied for all headings and any roughness.

Shapes, combinations of shapes or closely grouped non-structural items which do not readily fall into the above categories should be assessed from relevant literature^[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 site, depth and season. Where possible, site-specific or regional data should be used. If such data are not available, all members below MSL + 2 m should be considered to have a marine growth thickness equal to 12,5 mm (i.e. total of 25 mm across the diameter of a tubular member). In some areas of the world, this default thickness can be significantly exceeded.

The nominal sizes of structural members, conductors, risers, and appurtenances should be increased to account for the thickness of pre-existing and new marine growth. Marine growth on the teeth of elevating racks and protruding guided surfaces of chords can normally be ignored.

The marine growth thickness may be ignored if anti-fouling, cleaning or other means are applied. The surface roughness should still be taken into account, see A.7.3.2.4 or ISO/TR 19905-2:2012, A.7.3.2.4.

A.7.3.2.6 Hydrodynamic models for appurtenances

Raw water caissons on the legs and their guides should be included in the hydrodynamic model of the structure.

NOTE The guides for raw water caissons can cause a significant increase in the leg drag load, especially when they are comprised of high drag sections such as I-beams, flat bar, etc.

Depending upon the type and quantity, appurtenances can significantly increase the global wave actions. Appurtenances such as stairways, ladders and jetting lines should be considered for inclusion in the hydrodynamic model of the structure.

Appurtenances are generally modelled by means of increasing the effective diameter and/or hydrodynamic coefficients of a structural member.

A.7.3.3 Wave and current actions

A.7.3.3.1 General

Hydrodynamic actions for deterministic or stochastic analysis should be calculated using the Morison equation in combination with the hydrodynamic model and appropriate wave theories as described in the remainder of A.7.3.3. The wave and current velocities should be combined before they are used in the Morison equation. The intrinsic and apparent wave periods should be used appropriately; see A.7.3.3.5.

A.7.3.3.2 Hydrodynamic actions

Wave and current actions on slender members having cross-sectional dimensions sufficiently small compared with the wave length should be calculated using the Morison equation. The Morison equation is normally applicable providing that

$$\lambda > 5D_i \tag{A.7.3-12}$$

where

λ is the wave length;

D_i is the reference dimension of member (e.g. tubular diameter).

The Morison equation specifies the action per unit length as the vector sum as given in Equation (A.7.3-13):

$$\Delta F = \Delta F_{\text{drag}} + \Delta F_{\text{inertia}} = 0,5 \rho D C_D v_n |v_n| + \rho C_M A \dot{u}_n - \rho C_A A \ddot{r}_n \tag{A.7.3-13}$$

where the terms of the equation are described as follows.

To obtain the drag action, the appropriate drag coefficient (C_D) should be chosen in combination with a reference diameter, including any increase for marine growth, as described in A.7.3.2.

The Morison drag action formulation is as given in Equation (A.7.3-14):

$$\Delta F_{\text{drag}} = 0,5 \rho C_D D v_n |v_n| \tag{A.7.3-14}$$

where

ΔF_{drag} is the drag action (per unit length) normal to the axis of the member considered in the analysis and in the direction of v_n ;

ρ is the mass density of water (normally 1 025 kg/m³);

C_D is the drag coefficient (= C_{Di} or C_{De} from A.7.3);

v_n is the fluid particle velocity resolved normal to the member axis;

D is the reference dimension in a plane normal to the fluid velocity $v_n = D_i$ or D_e from A.7.3.

The fluid particle velocity, v_n , may either be the absolute or relative fluid particle velocity. In a deterministic analysis, the absolute fluid particle velocity is applied. In a stochastic analysis, the fluid particle velocity, v_n , may be taken as given in Equation (A.7.3-15):

$$v_n = u_n + V_{Cn} - \alpha \dot{r}_n \quad (\text{A.7.3-15})$$

where

$u_n + V_{Cn}$ is the combined particle velocity found as the vector sum of the wave particle velocity and the current velocity, normal to the member axis;

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

α = 0, if an absolute velocity is to be applied, i.e. neglecting the structural velocity;
 = 1, if relative velocity is being included. It may be used for stochastic/random wave action analyses only if the following applies:

$$u^* T_n / D_i \geq 20$$

where

u^* is the particle velocity = $V_C + \pi H_s / T_{z,i}$;

T_n is the first natural period of surge or sway motion;

D_i is the reference diameter of a chord.

NOTE See also A.10.4.3 for relevant damping coefficients depending on α .

To obtain the inertia action, the appropriate inertia coefficient (C_M) should be taken in combination with the cross-sectional area of the geometric profile, including any increase for marine growth, as described in A.7.3.2.3. The Morison's inertia action formulation is as given in Equation (A.7.3-16):

$$\Delta F_{\text{inertia}} = \rho C_M A \dot{u}_n - \rho C_A A \ddot{r}_n \quad (\text{A.7.3-16})$$

where

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

C_M is the inertia coefficient;

A is the cross-sectional area of member (equal to A_i or A_e from A.7.3.2);

\dot{u}_n is the wave particle acceleration normal to member;

C_A is the added mass coefficient, $C_A = C_M - 1$;

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

The last term in Equation (A.7.3-16) is not included in a deterministic analysis. The term should be included in a stochastic analysis representing the added mass force due to the member acceleration.

$$m_a \ddot{r}_n = \rho C_A A \ddot{r}_n \quad (\text{A.7.3-17})$$

where m_a is the added mass contribution (per unit length) for the member.

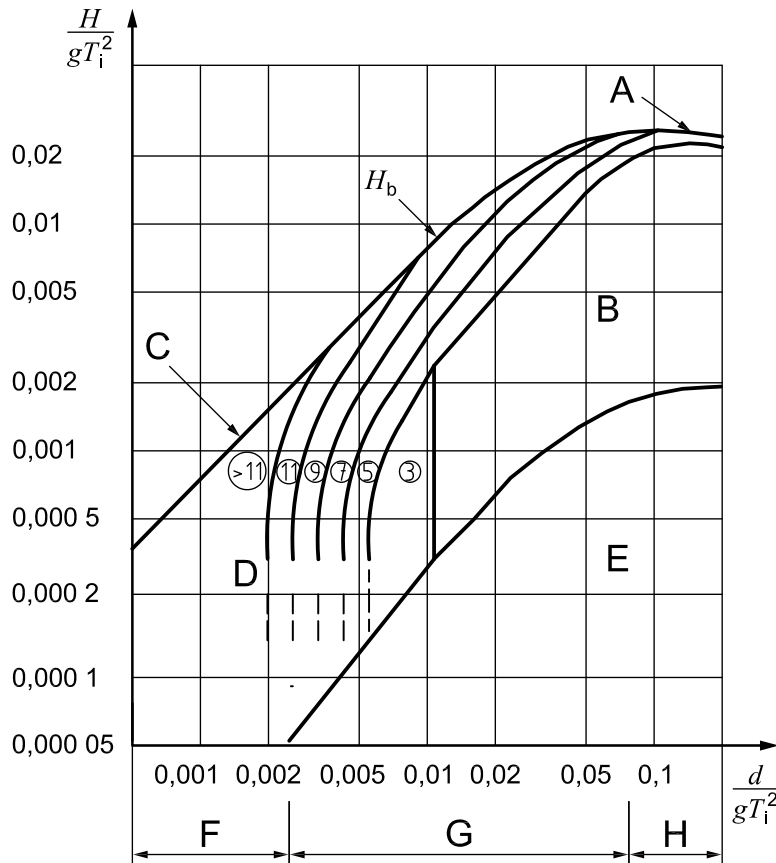
In a dynamic response analysis, the added mass (m_a integrated over the member length) is normally transferred to the left hand side of the equation of motion and added to the structural mass.

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 19901-1:2005, A.8.4, as shown in Figure A.7.3-5. For practical purposes, Stokes' 5th (within its bounds of applicability) or an appropriate order of Dean's Stream Function are acceptable for regular wave elevated storm analysis.

If breaking waves are indicated according to ISO 19901-1:2005, A.8.4, it is recommended that the wave period is changed to comply with the breaking limit for the specified height.



Key

- d* water depth
- g* acceleration due to gravity
- H* maximum wave height
- H_b* breaking wave height
- T_i* intrinsic wave period
- A deep water breaking limit $\ll H/\lambda = 0,14$
- B Stokes' fifth order, New-wave or third order stream function
- C shallow water breaking limit $H/d = 0,78$
- D stream function (showing order number)
- E linear/Airy or third order stream function
- F shallow water
- G intermediate depth
- H deep water

NOTE Taken from ISO 19901-1.

Figure A.7.3-5 — Regions of applicability of alternative wave theories

A.7.3.3.3.2 Stochastic waves

Time domain analysis is recommended for stochastic wave jack-up analysis. In such analyses the waves are modelled using a random superposition model to represent the wave spectrum; see A.6.4.2.5 to A.6.4.2.8. It is recommended that the random sea state be generated from the summation of at least 200 component waves of height and frequency determined to match the wave spectrum. The phasing of the component waves should be selected at random. A two-dimensional first order simulation using linear (Airy) waves is normally sufficient. However, when the effects of wave spreading is explicitly included in the analysis method, a three-dimensional simulation using a higher order wave theory should be used to capture higher frequency interaction effects (e.g. those due to frequency sum terms).

For first order wave kinematic models, the extrapolation of the wave kinematics to the free surface (wave stretching) is most appropriately carried out by substituting the true elevation at which the kinematics are required with one which is at the same proportion of the still water depth as the true elevation is of the instantaneous water depth. This can be expressed as given in Equation (A.7.3-18):

$$z' = \frac{z - \zeta}{1 + \zeta/d} \quad (\text{A.7.3-18})$$

where

- z' is the modified coordinate for use in particle velocity formulation;
- z is the vertical coordinate relative to the SWL under consideration, positive upwards, at which the kinematics are required;
- ζ is the instantaneous water level (same axis system as z);
- d is the water depth, still or undisturbed (positive).

This method ensures that the kinematics at the instantaneous free surface are always evaluated from the linear wave theory expressions as if they were at the still water level, see Reference [A.7.3-3] and ISO/TR 19905-2:2012, A.7.3.3.3.2.

For higher order wave-kinematic models, an appropriate alternative for stretching the wave profile to the instantaneous wave surface should be adopted.

The statistics of the underlying random wave process are Gaussian and fully known theoretically. The empirical modification around the free surface to account for free surface effects, together with the fact that drag actions are a non-linear (squared) transformation of wave kinematics, makes the hydrodynamic action excitation always non-linear. As a result, the random excitation is non-Gaussian. The statistics of such a process are generally not known theoretically, but the extremes are generally larger than the extremes of a corresponding Gaussian random process. For a detailed investigation of the dynamic behaviour of a jack-up, the non-Gaussian effects should be included. Multiple procedures for doing this are presented in Annex C.

When the random displacements of the submerged parts are small and the velocities are significant with respect to the water-particle velocities, the damping is not well represented by the relative velocity formulation in the Morison equation, which tends to overestimate the damping and underpredict the response. A criterion for determining the applicability of the relative velocity formulation is given in A.7.3.3.2.

A summary of recommendations for the time domain modelling of random waves is given in Table A.7.3-3.

Table A.7.3-3 — Recommendations for modelling of time domain stochastic waves

Method	Recommendations
Time domain	<p>Generate random sea from at least 200 components and use divisions of generally equal energy. It is recommended that smaller energy divisions be used in the higher frequency portion of the spectrum, which generally contains the reinforcement and cancellation frequencies. For each component, the relationship between wave length and frequency should be taken according to its linear dispersion relationship^[A.7.3-4].</p> <p>Unless indicated otherwise in the site-specific information, the validity of wave surface simulation should be checked against the criteria given below. The criteria for higher order waves should be taken to assure that H_s, mean waves and maximum crests are within practical limits.</p> <ul style="list-style-type: none"> — correct mean wave elevation; — standard deviation = $(H_s/4) \pm 1\%$; — $-0,03 < \text{skewness} < 0,03$; — $2,9 < \text{kurtosis} < 3,1$; — maximum crest elevation = $(H_s/4) \sqrt{2 \ln(N)}$ -5% to $+7,5\%$; <p>where N is the number of cycles in the time series being qualified, $N \approx \text{Duration}/T_z$.</p> <p>Integration time step less than the smaller of</p> <p style="text-align: center;">$T_z/20$ or $T_n/20$</p> <p>where</p> <ul style="list-style-type: none"> T_z is the apparent zero-upcrossing period of the wave spectrum; T_n is the jack-up natural period, see A.10.4.2.1 <p>(unless it can be shown that a larger time step leads to no significant change in results).</p> <p>Avoid transient effects, discard at least the first 100 s (the “run-in”).</p> <p>Ensure the simulation is of sufficient duration so that the method chosen results in demonstrably stable MPME responses; see also A.10.5.3.4 and Annex C.2.</p>

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

Both the magnitude of the actions on the structure and the dynamic amplification are affected by cancellation and reinforcement of wave actions, dependent on leg spacing (heading) and wave length. The effects of directionality and wave spreading should therefore be considered in any random dynamic analysis. The following two methods can be used to develop a representative DAF in conjunction with adjustments to the natural period (A.10.4.2.5.3).

Method 1: In a two-dimensional long-crested simulation, the effect of directionality can be included by developing a base shear transfer function (BSTF) accounting for spreading, “BSTF with spreading”, as described below (see 7.6.4 of Reference [A.7.3-4]).

- a) Develop a set of two-dimensional BSTFs, one for the “principal” direction of interest, and the others offset from the principal direction.
- b) For each offset direction, calculate a directionality contribution factor from ISO 19901-1:2005, A.8.7, or from ISO/TR 19905-2:2012, 6.4.2.8. Each factor corresponds to a given percentage of area under the directionality function such that the sum of all the factors is 1,0.
- c) The “BSTF with spreading” is then the sum of each two-dimensional BSTF (principal one plus the offset directions) multiplied by the corresponding directionality factors. Be aware that only the principal direction vector component of the offset direction BSTFs is used.
- d) The BSTF for the chosen two-dimensional (long-crested/unspread) analysis direction and the “BSTF with spreading” are compared to determine whether the selected direction is unconservative. Optimally, the direction of the two-dimensional sea state should be chosen to obtain a match with the three-dimensional BSTF for the entire wave frequency range. If this is not possible, the match between the spread and unspread BSTFs should be good at the natural period.

Method 2: To minimize reinforcement and cancellation effects, it is suggested that the dynamic analysis be carried out for a single wave heading along an axis that is neither parallel nor normal to a line through two adjacent leg centres. Thus, for a 3-legged jack-up with equilateral leg positions and a single bow leg, suitable analysis headings can be with the weather approaching from approximately 15° or 45° off the bow. The DAFs should be determined for one, or both, of these headings with suitably adjusted natural period; see Figure A.10.4-1. The DAFs (or more conservative DAFs) can then be applied to the final deterministic analysis for all headings.

A.7.3.3.4 Current

The current velocity and profile as specified in A.6.4.3 should be used. Where the current profile is defined by discrete points, linear interpolation between the data points is sufficient.

The current induced drag actions are determined in combination with the wave actions. This is carried out by the vectorial addition of the wave and current induced particle velocities prior to the drag action calculations.

The current velocity may be reduced to account for interference from the structure with the flow field of the current, as given in Equation (A.7.3-19); see Reference [A.7.3-5] and ISO/TR 19905-2:2012, 7.3.3.4:

$$V_C = V_f [1 + C_{De} D_e / (4D_F)]^{-1} \quad (\text{A.7.3-19})$$

where

V_C is the current velocity for use in the hydrodynamic model; V_C should not be taken as less than $0,7V_f$;

V_f is the far field (undisturbed) current velocity;

C_{De} is the equivalent drag coefficient of the leg, as defined in A.7.3.2;

D_e is the equivalent diameter of the leg, as defined in A.7.3.2;

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

A.7.3.3.5 Intrinsic and apparent wave periods

The intrinsic wave period is based on a reference frame travelling with the speed and direction of the current, and should be used, except as detailed later in this subclause, to calculate the wave kinematics. The apparent wave period is that which is observed by a stationary observer and is the period that should be used to calculate the jack-up dynamics. The intrinsic wave period, in conjunction with the water depth and appropriate wave theory, are used to calculate the wave length.

NOTE 1 There is only the intrinsic wave length; there is no apparent wave length. If one applies the apparent wave period in an analysis, the excitation period is correct but both the kinematics and the wave length are wrong. The wrong wave length means that the legs of a jack-up are at the wrong relative positions in the wave. The conceptual solution is to model the un-modified intrinsic wave with the jack-up moving into the wave at the current velocity.

It is important to determine whether the supplied wave period is apparent or intrinsic, taking due care to ensure that ISO 19901-1 terminology is consistently adhered to at all times. ISO 19901-1 terminology can conflict with the definition of these terms used by the supplier of the metocean data.

NOTE 2 ISO 19901-1 uses terminology conflicting from that in API RP 2A, Reference [A.7.3-6]. In ISO 19901-1, the “apparent” wave period is defined as the wave period seen by a stationary observer, while the “intrinsic” wave period is the wave period seen by an observer moving with the current. In API RP 2A the “actual” wave period is defined as the wave period seen by a stationary observer, while the “apparent” wave period is the wave period as it “appears” to an observer moving with the current. By comparison, ISO 19901-1 “intrinsic” equates to RP 2A “apparent”, and ISO 19901-1 “apparent” equates to RP 2A “actual”.

Equations for transformation between the intrinsic and apparent wave periods are given in ISO 19901-1:2005, A.8.3. It gives no direct guidance on modifying short crested sea states, although a suitable method can be inferred. The assessor should ensure that the correct procedure is used by the software in calculating wave particle kinematics and dynamics; it is important to understand the terminology used by the software vendor; see Note above. In summary, the steps taken to convert intrinsic to apparent wave period are as follows.

- a) Calculate the wave length based on the intrinsic wave period and the water depth, using a suitable wave theory.
- b) Calculate the intrinsic wave celerity as wave length divided by intrinsic wave period.
- c) Calculate the apparent wave celerity by adding the resolved current velocity to the wave celerity (the celerity is increased if the current is in the same direction as wave propagation, and decreased if in an opposing direction).
- d) Calculate the apparent wave period as the wave length divided by the apparent celerity.

Conversion from an apparent wave period to an intrinsic wave period follows a similar approach, but is undertaken iteratively.

Care should be taken with opposing currents that the vector sum of apparent celerity and current is always greater than or equal to zero, otherwise the waves move backwards. This is likely to be relevant only for very short period waves when developing the apparent component periods of a random seastate.

This conversion procedure between apparent and intrinsic periods strictly applies in the case of simple uniform currents over the full water depth. It can be used practically if the current is uniform over the top 50 m of the water column. In cases of a non-uniform current profile, a weighted, depth-averaged in-line current speed, $V_{IN-LINE}$, may be used, as shown in ISO 19901-1:2005, A.8.3, and Reference [A.7.3-7] and as given in Equation (A.7.3-20):

$$V_{IN-LINE} = \frac{2k}{\sinh(2kd)} \int_{-d}^0 V_c(z) \cos \theta(z) \cosh[2k(z+d)] dz \quad (A.7.3-20)$$

where

- k is the wave number = $2\pi/\lambda$;
- λ is the actual wave length (i.e., deepwater wave length corrected for water depth);
- d is the water depth;
- $V_c(z)$ is the current velocity at depth z ;
- z is the vertical coordinate relative to the SWL under consideration, positive upwards;
- $\theta(z)$ is the angular direction of the current at depth z relative to the wave propagation direction;
 $\theta(z) = 0,0$ when in line.

In a two-stage analysis the deterministic quasi-static wave/current actions should be determined using the intrinsic period.

The apparent wave period should be used for the SDOF DAF calculation of $K_{DAF,SDOF}$.

For stochastic calculations, the rigorous approach is to develop the particle kinematics for the components using the intrinsic wave period and to develop the wave/current actions by applying the intrinsic kinematics to the jack-up by using component wave phases based on the apparent wave period. This approach should be used for one-stage analysis and for two-stage analysis with a non-linear foundation model for the DAF calculations. This procedure is difficult if the available analytical tools do not have the feature implemented.

When undertaking a two-stage deterministic storm analysis (A.10.5.2) using a DAF developed from a random dynamic analysis (A.10.5.2.2.3) with linearized foundations, it can be acceptable to use a spectrum with an apparent peak period for all stages in the calculation of $K_{\text{DAF,RANDOM}}$ and the inertial loadset. The error is expected to be small when the ratio $T_{p,i}/T_p$ is within the range $1 \pm 0,08$. If this approach is used, the analysis should also be undertaken without period adjustment and the more onerous DAFs used. When $T_{p,i}/T_p$ is outside this range, a more rigorous approach should be considered.

A.7.3.4 Wind actions

A.7.3.4.1 Wind action

The wind action on each component (divided into blocks of not more than 15 m vertical extent), F_{Wi} , can be computed using Equation (A.7.3-21):

$$F_{Wi} = P_i A_{Wi} \quad (\text{A.7.3-21})$$

where

P_i is the pressure at the centre of block i ;

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

The pressure P_i should be computed using Equation (A.7.3-22):

$$P_i = 0,5 \rho V_{zi}^2 C_s \quad (\text{A.7.3-22})$$

where

ρ is the density of air (taken as 1,222 4 kg/m³ unless an alternative value can be justified for the site);

V_{zi} is the specified wind velocity at the centre of block i ; see A.6.4.5.2;

C_s is the shape coefficient, as given in A.7.3.4.2.

Wind actions on legs below the hull should be calculated to either the instantaneous wave surface or to SWL.

NOTE The wind area of the hull and associated structures (excluding derrick and legs) can normally be taken as the projected area viewed from the wind direction under consideration.

A.7.3.4.2 Shape coefficient

Using building block elements, the shape coefficients in Table A.7.3-4 should be used.

Table A.7.3-4 — Shape coefficients

Type of member or structure	Shape coefficient, C_s
Hull side (flat side)	1,0 based on total projected area
Hull and associated structures (excluding derrick and legs)	1,1 based on the total projected area (i.e. the area enclosed by the extreme contours of the structure)
Deckhouses, jack-frame structure, sub-structure, draw-works house, and other above-deck blocks	1,1 based on the projected area
Leg sections projecting above jack-frame structure and below the hull	$C_s = C_{De}$ as determined from A.7.3.2.3, normally using smooth drag coefficients (ignoring marine growth) A_{Wi} determined from D_e and section length
Isolated tubulars (crane pedestals, etc.)	0,5
Isolated structural shapes (angles, channels, box, I-sections)	1,5 based on member projected area
Derricks, crane booms, flare towers (open lattice sections only, not boxed-in sections)	The appropriate shape coefficient for the members concerned applied to 50 % of the total projected profile area of the item (25 % from each of the front and back faces)
Shapes or combinations of shapes that do not readily fall into the above categories should be subject to special consideration.	

A.7.3.4.3 Wind tunnel data

Wind pressures and resulting actions for the hull and associated structures may be determined from wind tunnel tests on a representative model. Care should be exercised when interpreting wind tunnel data for structures mainly comprised of tubular components, such as truss legs.

A.7.4 Functional actions

Provided appropriate procedures exist and it is practical to change the mode of the jack-up from operating to elevated storm mode on receipt of an unfavourable weather forecast, it is necessary to assess only the elevated storm mode. Consideration should be given to actions on the conductors if supported by the jack-up.

The following should be defined:

- a) actions due to the maximum and minimum elevated weight. In the absence of other information the minimum elevated weight can normally be determined assuming 50 % of the variable load permitted by the operating manual;
- b) extreme limits of the centre of gravity position (or reactions of the elevated weight on the legs) for the configurations in a) above;
- c) substructure and derrick position, hook load, rotary load, setback and conductor tensions for the configurations in a) above;
- d) weight, centre of gravity and buoyancy of the legs.

If a minimum elevated weight or a limitation of the centre of gravity position is required to meet the overturning acceptance criteria (see 5.4.4 and 13.8), then the addition of water in lieu of variable load is permitted in the assessment, provided that

- the functional actions do not exceed the operations manual limits;
- procedures, equipment and instructions exist for performing the operation of adding water offshore;
- the action due to the maximum variable load, including added water, is used for all appropriate assessment checks (preload, stress, etc.).

If a reduction in elevated weight or a limitation of the centre of gravity position is required to meet the foundation acceptance criteria with respect to foundation sliding, see 5.4.4 and 13.9.1, then the variable load used in the assessment can be revised accordingly provided that procedures, equipment and instructions exist for the timely performance of the operation offshore.

A.7.5 Displacement dependent actions

No guidance is offered.

A.7.6 Dynamic effects

No guidance is offered.

A.7.7 Earthquakes

No guidance is 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

Techniques for modelling the legs, hull, leg-to-hull connection, and leg/spudcan connection are discussed. The leg-to-hull connection model includes the upper and lower guides, jacking pinions, fixation systems, and jackcase/associated bracing. Modelling of the foundation is limited to the structural details in this clause; geotechnical aspects are presented in A.9.

Because of the interaction of the mass and stiffness models, e.g. the effect of mass modelling on hull sag, it is recommended that the assessor be familiar with the whole of this clause before commencing the modelling.

A.8.2 Overall considerations

A.8.2.1 General

No guidance is offered.

A.8.2.2 Modelling philosophy

The structural model should accurately reflect the complex mechanism of the jack-up; for most jack-up configurations this requires the use of an FE computer model. A.8.3 to A.8.5 describe the structural aspects of the model. A.8.6 describes the interaction of the structural model with the foundation. A.8.7 describes modelling the mass and A.8.8 describes the application of the actions.

A.8.2.3 Levels of FE modelling

While it can be desirable to fully model the jack-up when assessing its structural strength, this is rarely necessary for a site-specific assessment. An overly complex model can introduce errors and unnecessarily complicate the assessment. Consequently assumptions and simplifications, such as equivalent hull,

equivalent leg, etc., are often made when building the model(s) used for the assessment. In view of this, one of the various levels of modelling described in a) through d) below can be used. It should be recognized that some of these methods have limitations with respect to the accuracy of assessing the structural adequacy of a jack-up. Table A.8.2-1 outlines the limitations of the various modelling techniques and should be referenced to ensure that the selected model addresses all aspects required for the assessment. When simplified models, such as those described in b) and d) are used, it is usually appropriate to calibrate them against a more detailed model.

a) Fully detailed leg model:

The model consists of “detailed legs”, hull, leg-to-hull connections and spudcans modelled in accordance with A.8.3.2, A.8.4, A.8.5 and A.8.6, respectively. The results from this model can be used to examine all aspects of a jack-up site-specific assessment, including foundation stability, overturning resistance, leg strength and the adequacy of the jacking system or fixation system.

b) Equivalent leg (stick model):

The model consists of “stick model” legs (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-to-hull connection model d).

c) Combined equivalent/detailed leg and hull model:

The model consists of a combination of “detailed leg” for the upper portion of legs and “stick model” for the lower portion of the legs (A.8.3.4). The hull, leg-to-hull connections and spudcans are modelled in accordance with A.8.4, A.8.5 and A.8.6, respectively. The results from this model can be used to examine foundation stability, overturning resistance, leg strength in the region of the leg-to-hull connections and the adequacy of the jacking and/or fixation systems. See Figure A.8.2-1.

d) Detailed single leg and leg-to-hull connection model:

The model consists of a “detailed leg” or a portion of a “detailed leg” (A.8.3.2), the leg-to-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.

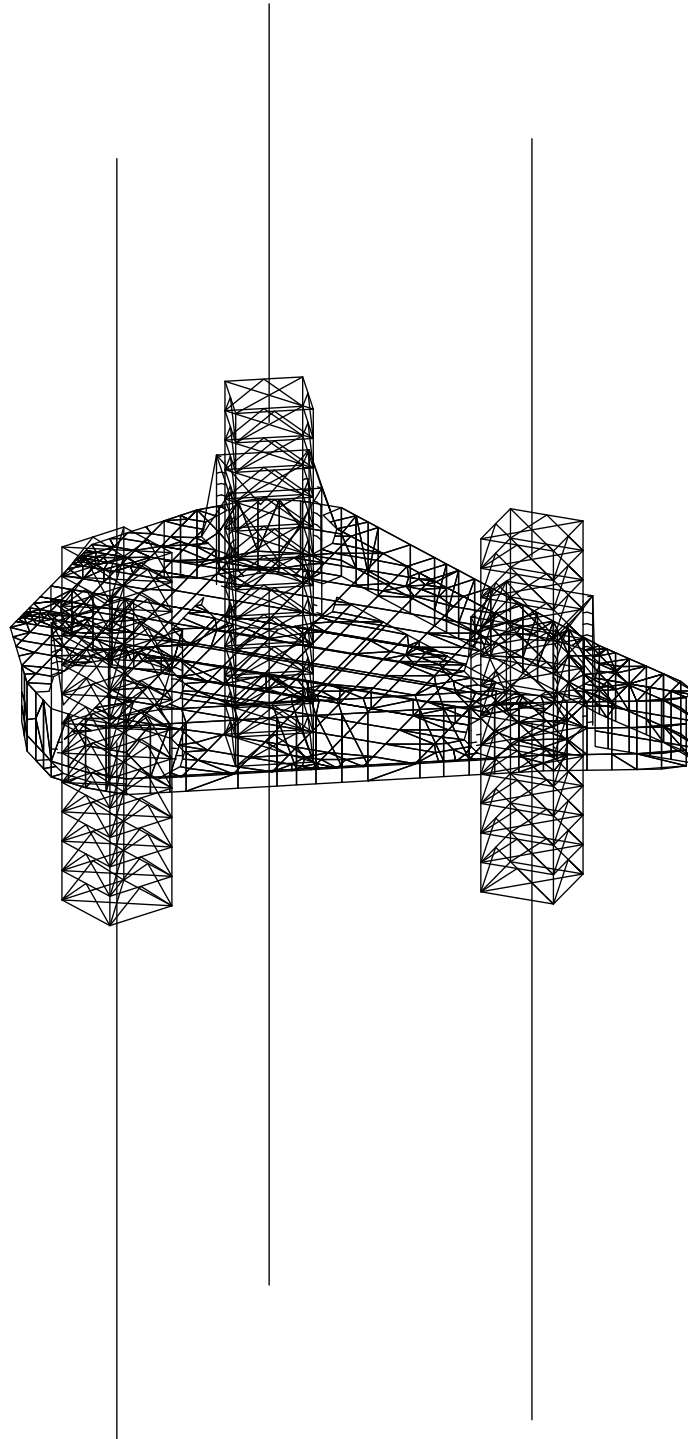


Figure A.8.2-1 — Combined equivalent/detailed leg and hull model

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

Model type	Applicability						
	I Base shear and overturning moment	II Overturning checks	III Foundation checks	IV Global leg forces	V Leg member forces	VI Jacking/fixation system reactions	VII Hull element forces
a) Fully detailed leg	Yes	Yes	Yes	Yes	Yes	Yes	See note
b) Equivalent leg (stick model)	Yes	Yes	Yes	Yes	—	—	—
c) Combined equivalent/detailed leg and hull	Yes	Yes	Yes	Yes	Yes	Yes	See note
d) Detailed single leg and leg-to-hull connection model	—	—	—	—	Yes	Yes	—

NOTE Hull stresses are only available from more complex hull models.

A.8.3 Modelling the leg

A.8.3.1 General

For truss legs the model(s) can be generated in accordance with A.8.3.2 to A.8.3.4 as applicable. Single column legs can be modelled with beam elements (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 equations indicated in Tables A.8.3-1 and A.8.3-2 or derived from the application of suitable unit load cases to the 'detailed leg'. The stiffness properties of the equivalent leg should equate to those of the 'detailed leg' model described in A.8.3.2. Where such a model is used, relevant analysis results can be applied to a detailed leg model to determine member stresses, fixation system/pinion forces, etc.

The determination of stiffness for the equivalent leg model can be accomplished as outlined below.

- a) From hand calculations using the equations presented in Tables A.8.3-1 and A.8.3-2. If the leg scantlings change in different leg sections, this can be accounted for by calculating the properties for each leg section and creating the equivalent leg model accordingly. Provided that there are no significant offsets between the brace work points, these are reasonably accurate for cases A (sideways K bracing), C (X bracing) and D (Z bracing). Case B (normal K bracing) should be used with caution as the values of equivalent shear area and second moment of area are dependent on the number of bays being considered.

- b) From the application of unit load cases to a detailed leg model prepared in accordance with 8.3.2 and 8.3.5: The leg should be rigidly restrained, generally at the first point of lateral force transfer between the hull and the leg, although it can be more convenient to use a different reference point, e.g. level of the fixation system or neutral axis of the hull. The variables Δ , δ , θ_M and θ_P used in Equations (A.8.3-1) to (A.8.3-4) are obtained from the detailed leg model. The following load cases should be considered, applied about the major and minor axes of the leg:

- Axial unit load case: This is used to determine the axial area, A , of the equivalent beam according to standard beam theory as given in Equation (A.8.3-1):

$$\Delta = \frac{FL}{AE} \Rightarrow A = \frac{FL}{E\Delta} \quad (\text{A.8.3-1})$$

where

- Δ is the axial deflection (shortening) of the cantilever at the point of force application;
- F is the applied axial action;
- L is the cantilevered length from the hull to the seabed reaction point; see A.8.6.2;
- E is Young's modulus.

- Pure moment applied either as a moment or as a couple at the end of the cantilever: This is used to derive the second moment of area (I) according to standard beam theory as given in Equations (A.8.3-2):

$$\delta = \frac{ML^2}{2EI} \Rightarrow I = \frac{ML^2}{2E\delta} \quad \text{and} \quad \theta_M = \frac{ML}{EI} \Rightarrow I = \frac{ML}{E\theta_M} \quad (\text{A.8.3-2})$$

where

- δ is the lateral deflection of the cantilever at the point of moment application;
- M is the applied moment;
- θ_M is the slope of the cantilever at the point of moment application.

It should be recognized that the value of I resulting from the two equations can differ somewhat.

- Pure shear, P , applied at the end of the cantilever, which can be used to derive I according to standard beam theory as given in Equation (A.8.3-3):

$$\theta_P = \frac{PL^2}{2EI} \Rightarrow I = \frac{PL^2}{2E\theta_P} \quad (\text{A.8.3-3})$$

where

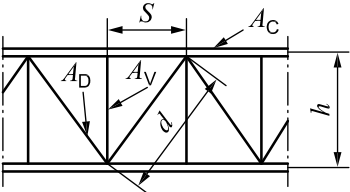
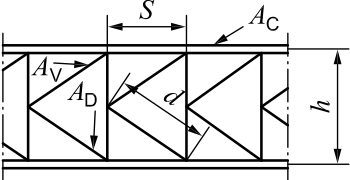
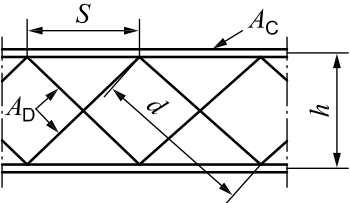
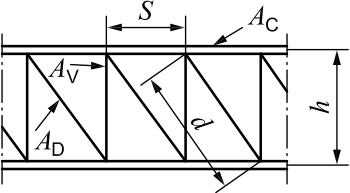
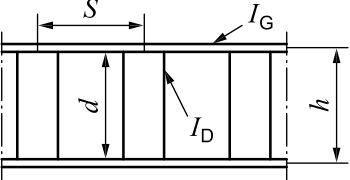
- P is the applied shear;
- θ_P is the slope of the cantilever at the point of shear application.

Using either this value of I , or a value obtained from the pure moment case, the effective shear area, A_s , can then be determined using Equation (A.8.3-4):

$$\delta = \frac{PL^3}{3EI} + \frac{PL}{A_s G} \Rightarrow A_s = \frac{7,8PLI}{3EI\delta - PL^3} \quad (\text{A.8.3-4})$$

where G is the shear modulus of steel, $G = E/2,6$ for Poisson's ratio of 0,3.

Table A.8.3-1 — Equations for determining the effective shear area for two dimensional structures

	Structure	Effective shear area
A		$A_{si} = \frac{(1+\nu)sh^2}{\frac{d^3}{2A_D} + \frac{s^3}{6A_C}}$
B		$A_{si} = \frac{(1+\nu)sh^2}{\frac{d^3}{A_D} + \frac{h^3}{SA_V} - \frac{s^3}{NA_C} \left(\frac{N^3}{3} - \sum_{i=1}^N i^2 \right)}$
C		$A_{si} = \frac{(1+\nu)sh^2}{\frac{d^3}{4A_D} - \frac{s^3}{12A_C}}$
D		$A_{si} = \frac{(1+\nu)sh^2}{\frac{d^3}{2A_D} + \frac{h^3}{2A_V} + \frac{s^3}{6A_C}}$
E		$A_{si} = \frac{48(1+\nu)I_G}{S^2 \left(1 + \frac{d}{S} \frac{I_G}{I_D} \right) \frac{d}{h}}$

Key

- S bay height
- h centre to centre of chords on face
- d length of diagonal brace on face
- A_C area of chord
- A_D area of brace diagonal
- A_V area of brace horizontal
- ν Poisson's ratio (0,3 for steel)
- I_G largest inertia of chord
- I_B largest inertia of brace
- N number of active bays

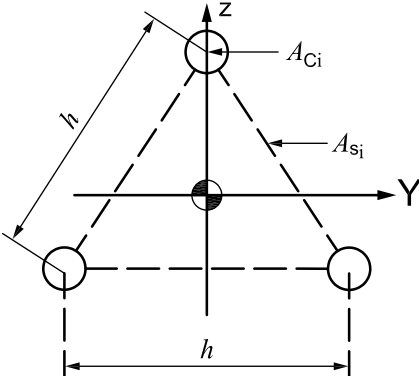
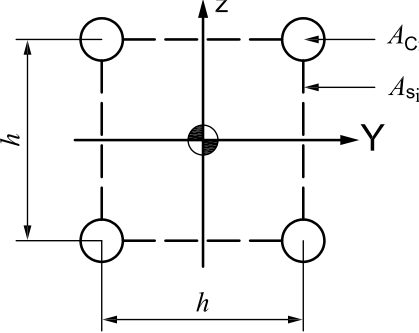
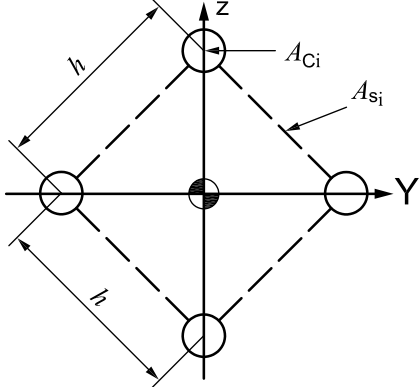
NOTE 1 The stiffness properties are the same for all directions unless the chords have different areas.

NOTE 2 The equations can be inaccurate if significant offsets exist between brace work points.

NOTE 3 The equivalent beam end rotations can be inaccurate for bracing type C. This can be important if this modelling is used in conjunction with rotational foundation stiffness.

NOTE 4 Based on DNV Class Note 31.5, 1992^[A.8.3-1], corrected.

Table A.8.3-2 — Equations for determining the equivalent section properties of three-dimensional lattice legs

	Leg type	Equivalent properties
A		$A = 3A_{Ci}$ $A_{sy} = A_{sz} = \frac{3}{2}A_{si}$ $I_y = I_z = \frac{1}{2}A_{Ci}h^2$ $I_T = \frac{1}{4}A_{si}h^2$
B		$A = 4A_{Ci}$ $A_{sy} = A_{sz} = 2A_{si}$ $I_y = I_z = A_{Ci}h^2$ $I_T = A_{si}h^2$
C		$A = 4A_{Ci}$ $A_{sy} = A_{sz} = 2A_{si}$ $I_y = I_z = A_{Ci}h^2$ $I_T = A_{si}h^2$

Key

A_{si} effective shear area for two-dimensional structure (from Table A.8.3-1)

A_{Ci} individual chord area

A_s effective shear area about representative axis (y or z)

I second moment of area about representative axis (y or z)

I_T torsional moment of inertia

NOTE 1 A_{Ci} can be taken as the cord area including a contribution from the rack teeth (see 8.3.5).

NOTE 2 Based on DNV Class Note 31.5, 1992^[A.8.3-1], corrected.

A.8.3.4 Combination of detailed and equivalent leg

The combined detailed and equivalent leg model should be constructed with the areas of interest modelled in detail and the remainder of the leg modelled as an equivalent leg. To facilitate obtaining detailed stresses in the vicinity of the leg-to-hull connection (guides, fixation/jacking system, etc.), the detailed portion of the leg model should extend far enough above and below this region to ensure that boundary conditions at the 'detailed leg'/equivalent leg' connection do not affect stresses in the areas of interest. Care should be taken to ensure an appropriate interface and consistency of boundary conditions at the connections.

The plane of connection between the "detailed leg" and the "equivalent leg" should remain a plane and without shear distortion when the leg is bent. The connection should be composed of rigid elements that control local bending and shear distortion.

A.8.3.5 Stiffness adjustment

No guidance is offered.

A.8.3.6 Leg inclination

No guidance is offered.

A.8.4 Modelling the hull

A.8.4.1 General

Recommended methods of modelling the hull structure are given in A.8.4.2 and A.8.4.3. Hull mass modelling is discussed in A.8.7 and the modelling of hull sagging is discussed in A.8.8.3.

A.8.4.2 Detailed hull model

The model should be generated using plate elements in which appropriate directional modelling of the effect of the stiffeners on the plates should be included. The elements should be capable of carrying in-plane shear and out-of-plane moment.

A.8.4.3 Equivalent hull model

In an equivalent hull model, the deck, bottom, side shell and major bulkheads are modelled as a grillage of beams. The axial and out-of-plane properties of the beams should be calculated based on the depth of the bulkheads, side shell and the "effective width" of the deck and bottom plating. Beam elements should be positioned with their neutral axes at mid-depth of the hull. Due to the continuity of the deck and bottom structures and the dimensions of a typical hull box, the in-plane bending stiffness can be treated as large relative to the out-of-plane stiffness. The torsional stiffness should be approximated from the closed box section of the hull and distributed between the grillage members.

A.8.5 Modelling the leg-to-hull connection

A.8.5.1 General

The leg-to-hull connection modelling is of extreme importance to the analysis since it controls the distribution of leg bending moments and shears carried between the upper and lower guide structures and the jacking or fixation system. It is, therefore, necessary that these systems be properly modelled in terms of stiffness, orientation and clearance. A simplified derivation of the equivalent leg-to-hull connection stiffness can be used for the equivalent leg (stick model).

A specific jack-up design concept can be described by a combination of the following components (see also Figure C.1-1):

- a) with or without fixation system;
- b) with opposed jacking pinions [see Figure A.8.5-1 a)];
- c) with unopposed jacking pinions [see Figure A.8.5-1 b)];
- d) with pin and yoke jacking system [see Figure A.8.5-1 c)];
- e) with fixed or floating jacking system.

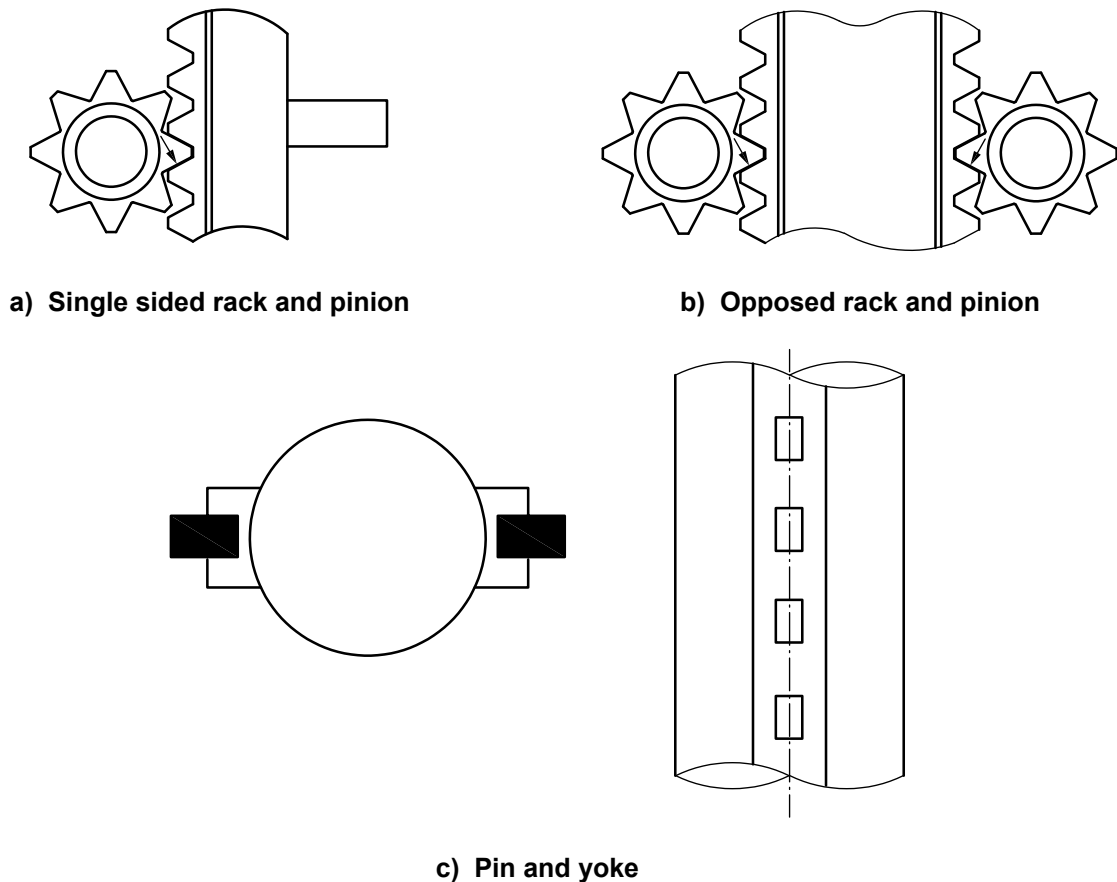
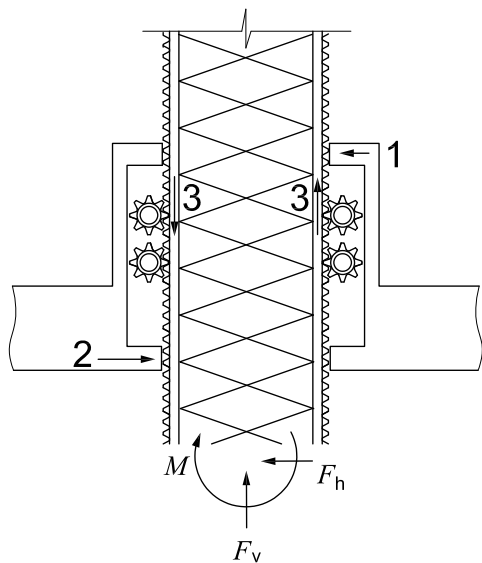


Figure A.8.5-1 — Types of elevating system

Representative leg-to-hull connections are shown in Figure A.8.5-2 a) through Figure A.8.5-2 d). The basic function of the leg-to-hull connection is to transfer forces between the leg and hull as follows.

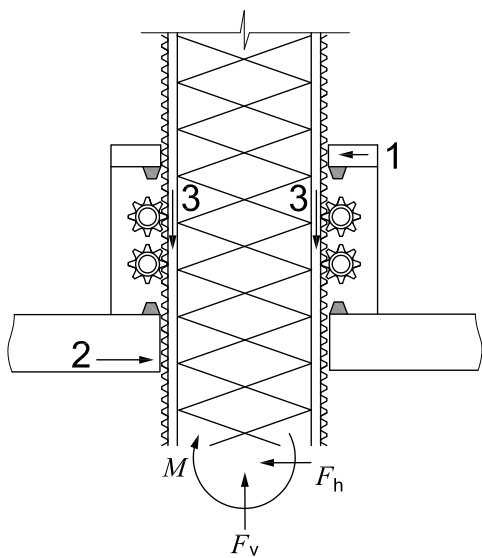
- Horizontal shear is transferred by a set of horizontal forces in the lower guides and/or fixation system.
- Vertical force is transferred via a set of vertical forces in the support system.
- Bending moment is transferred by a combination of horizontal forces in the upper and lower guides and/or by a set of vertical forces in the support system.



a) Fixed jacking system without fixation system

System includes

- jackcase;
- fixed jacking system with opposed or unopposed jacking pinions

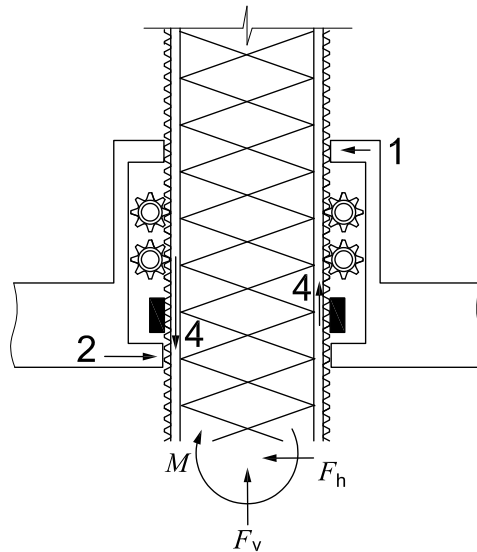


b) Floating jacking system without fixation system

System includes

- jackcase;
- shockpads;
- floating jacking system with opposed or unopposed jacking pinions

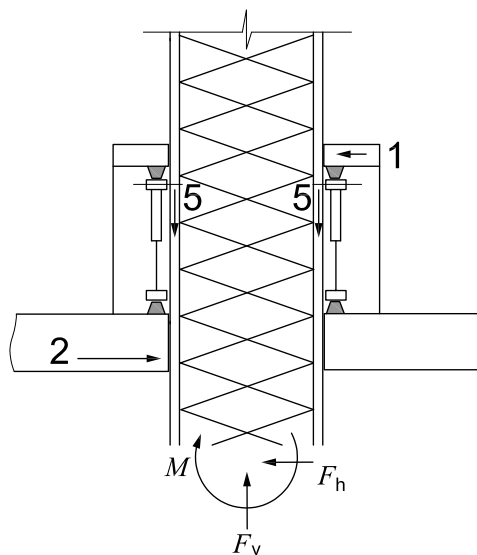
Figure A.8.5-2 (continued)



System includes

- jackcase;
- jacking system with opposed or unopposed jacking pinions;
- fixation system

c) Jacking system with fixation system



System includes

- jackhouse;
- upper and lower yokes;
- upper and lower shockpads;
- jacking cylinders;
- jacking pins

d) Pin and yoke jacking system

Key

- 1 upper guide reaction
- 2 lower guide reaction
- 3 pinion reactions
- 4 fixation system reactions
- 5 jacking pin reactions
- F_v axial force in leg at lower guide
- F_h shear force in leg at lower guide
- M bending moment in leg at lower guide

Figure A.8.5-2 — Representative leg-to-hull connections

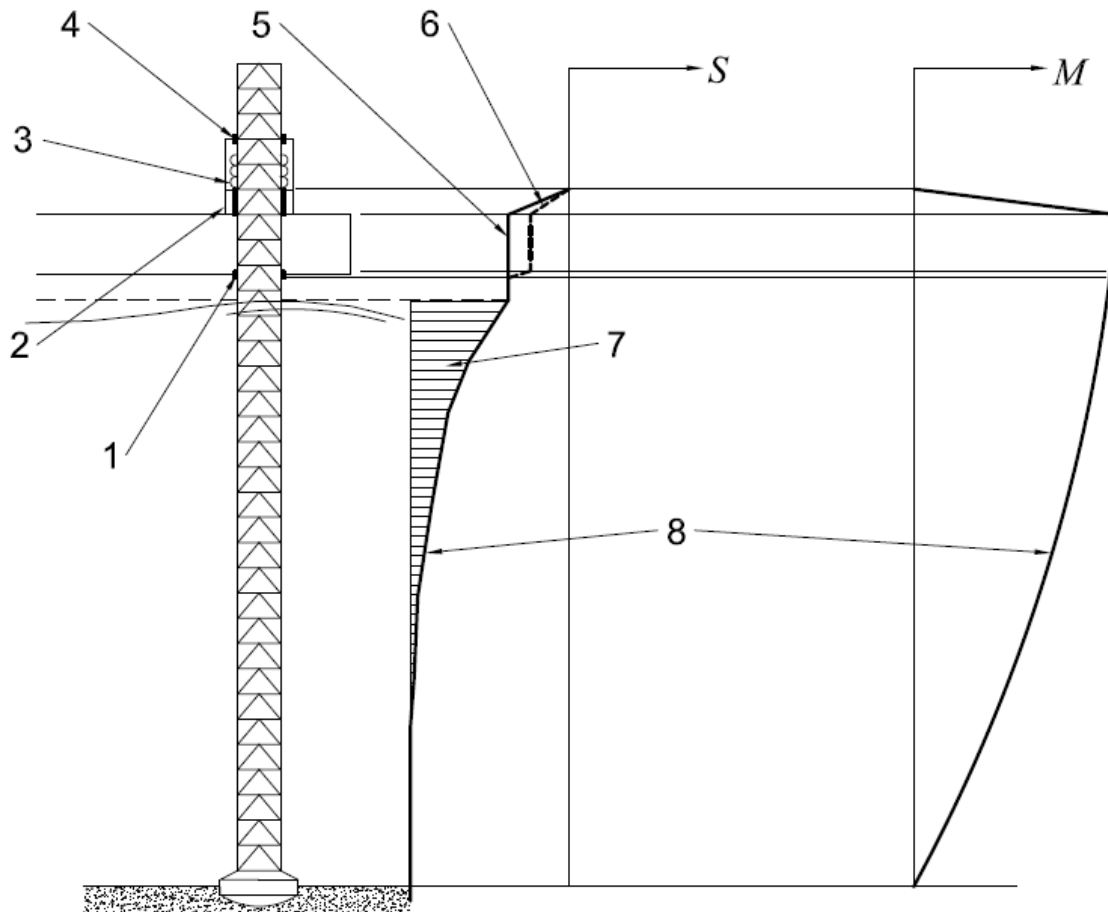
For jack-ups with a fixation system, the leg bending moment is shared by the upper and lower guides, the jacking system and the fixation systems. Normally, the leg bending moment and the axial force at the leg-to-hull connection due to the environmental actions are transferred largely by the fixation system because of its high stiffness. Depending on the specified method of operation, the stiffnesses, the initial clearances and the magnitude of the applied forces, a portion of the environmental leg loading can also be transferred by the jacking system and the guide structures. After the fixation systems are engaged, some jack-ups release the

pinions by disengaging the jacking system. Under this condition, the leg bending moment is shared by the upper and lower guides and the fixation systems. A complete typical shear force and bending moment diagram is shown in Figure A.8.5-3, with a more detailed representation shown in Figure A.8.5-4 a). The diagram below the lower guide is independent of the leg-to-hull connection.

For jack-ups without a fixation system, the leg bending moment is shared by the jacking system and guide structure. For jack-ups with a fixed jacking system, the distribution of leg moment between the jacking system and guide structure mainly depends on the stiffness of the jacking pinions. Typical shear force and bending moment diagrams for this configuration are shown in Figures A.8.5-4 b) and A.8.5-4 c).

For a floating jacking system, the distribution of leg bending moment between the jacking system and guide structure depends on the combined stiffness of the shock pads and pinions. Typical shear force and bending moment diagrams for this configuration are shown in Figure A.8.5-4 d).

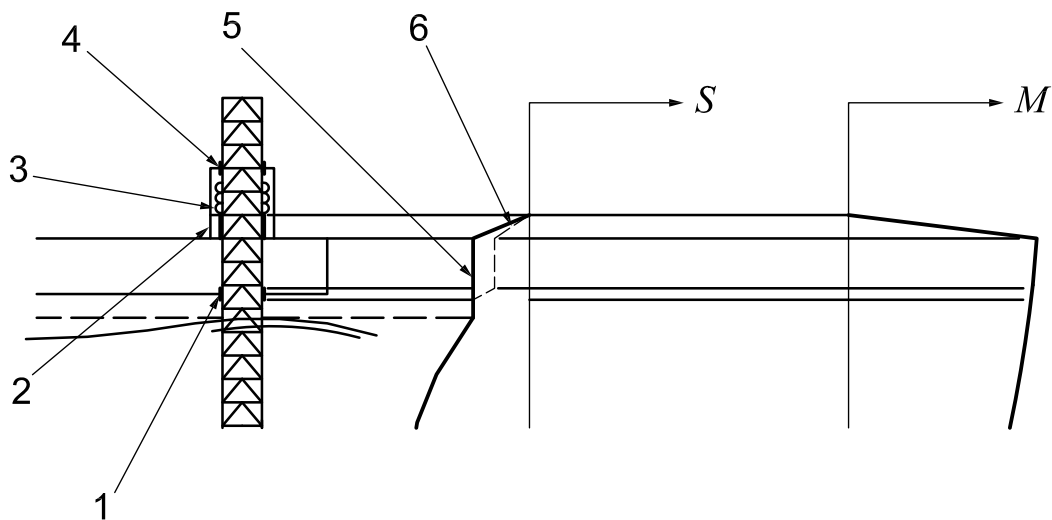
The leg-to-hull connection should be modelled considering the effects of guide and support system clearances, wear, construction tolerances and backlash (within the gear train and between the drive pinion and the rack).



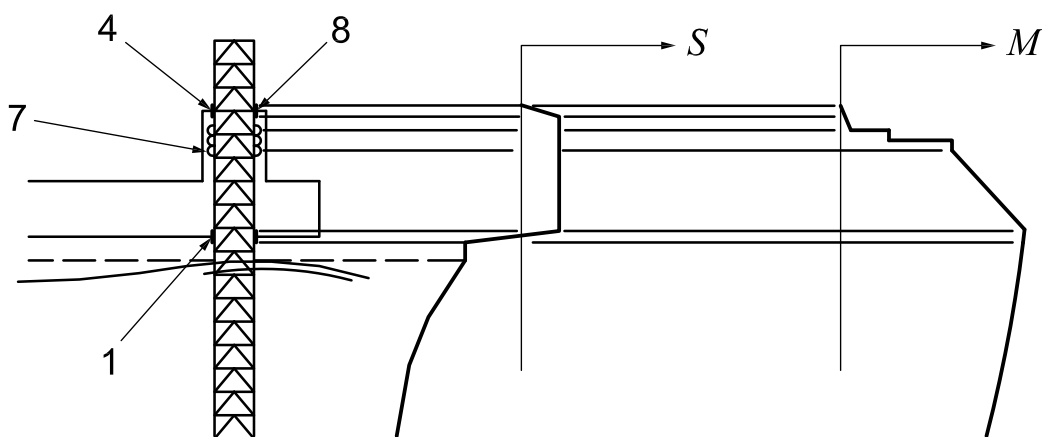
Key

- | | | | |
|----------|-----------------------|---|---|
| 1 | lower guide | 5 | shear force without lower guide contact |
| 2 | fixation system lower | 6 | shear force with lower guide contact |
| 3 | jacking pinion | 7 | shear due to wave/current action |
| 4 | upper guide | 8 | net shear or bending moment |
| <i>S</i> | shear force | | |
| <i>M</i> | bending moment | | |

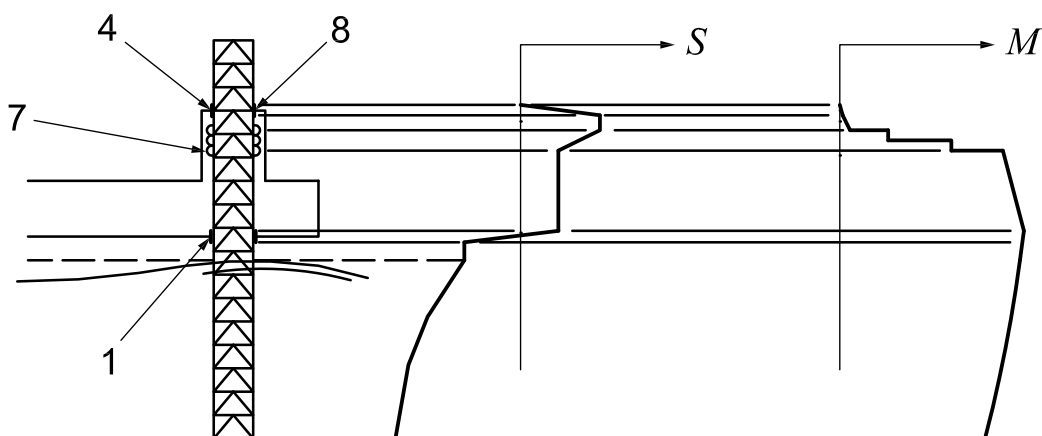
Figure A.8.5-3 — Complete leg shear force and bending moment — Jack-ups with a fixation system



a) Jack-ups with a fixation system

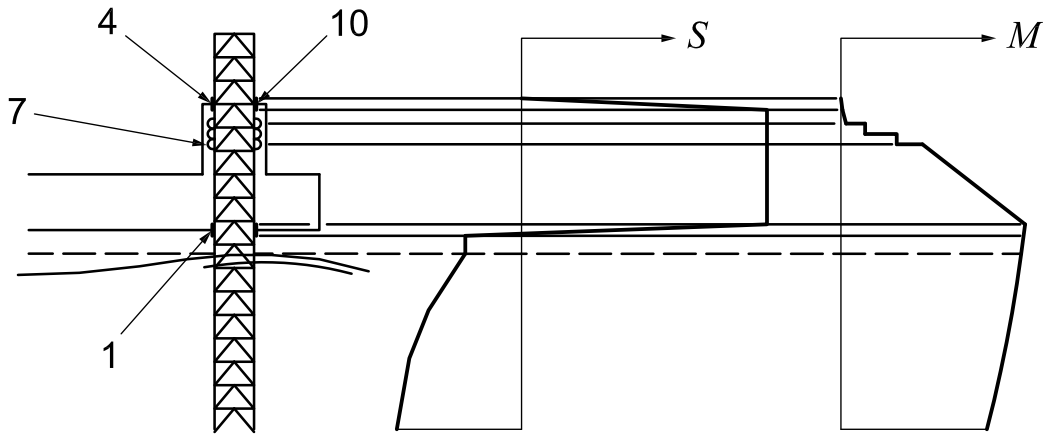


b) Jack-ups without a fixation system and having a fixed jacking system with opposed pinions



c) Jack-ups without a fixation system and having a fixed jacking system with unopposed pinions

Figure A.8.5-4 (continued)



d) Jack-ups without a fixation system and having a floating jacking system

Key

- | | | | |
|---|---|----|--------------------------------------|
| 1 | lower guide | 6 | shear force with lower guide contact |
| 2 | fixation system lower | 7 | opposed pinions |
| 3 | jacking pinion | 8 | jack case rigidly fixed to hull |
| 4 | upper guide | 9 | unopposed pinions |
| 5 | shear force without lower guide contact | 10 | jack case floating on shock pads |
- S* shear force
M bending moment

Figure A.8.5-4 — Leg shear force and bending moment within the leg-to-hull connection

If the jacking system has unopposed pinions, local chord moments arise due to

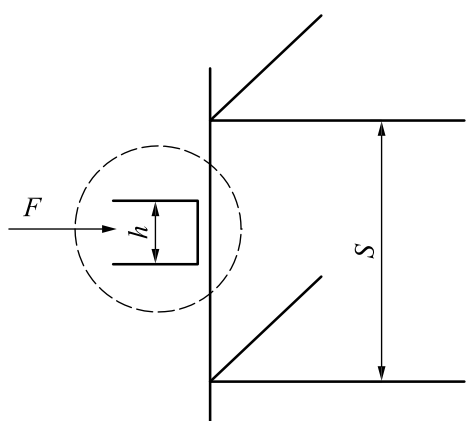
- the horizontal pinion force component (due to the pressure angle of the rack/pinion);
- the vertical pinion force component acting at an offset from the chord neutral axis.

The techniques in A.8.5.2 to A.8.5.7 are recommended for modelling leg-to-hull connections (specific data for the various parts of the structure can be available from the design data package).

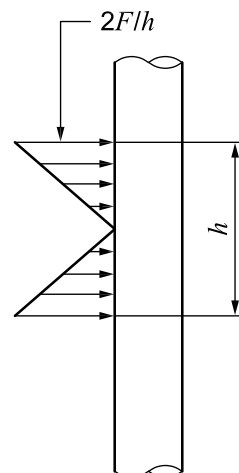
A.8.5.2 Guide systems

The guide structures should be modelled to restrain the chord member horizontally only in directions in which guide contact occurs. The upper and lower guides can be considered to be relatively stiff with respect to the adjacent structure, such as jackcase, etc. The nominal lower guide position relative to the leg can be derived using the sum of leg penetration, water depth and hull elevation. It is, however, recommended that at least two positions be covered when assessing leg strength: one at a node and the other at midspan. This is to allow for uncertainties in the prediction of leg penetration and possible differences in penetration between the legs.

The finite lengths of the guides can be included in the modelling by means of a number of discrete restraint springs/connections to the hull. Care should be taken to ensure that such restraints carry reactions only in directions/senses in which they can act. Alternatively, the results from analyses ignoring the guide length can be corrected, if necessary, by modification of the local bending moment diagram to allow for the proper distribution of guide reaction; see Figure A.8.5-5. The bending moments in the chord members at the guides determined from a finite element analysis ignoring the guide length, as in Figure A.8.5-5 a)i) and b)i), can be corrected using beam analysis for the simplified guide reactions, as shown in Figure A.8.5-5 a)ii) and b)ii).

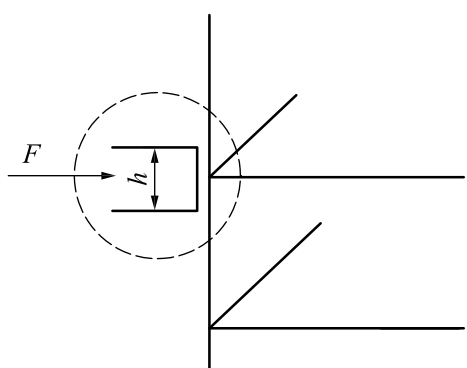


i) Guide reaction ignoring guide length

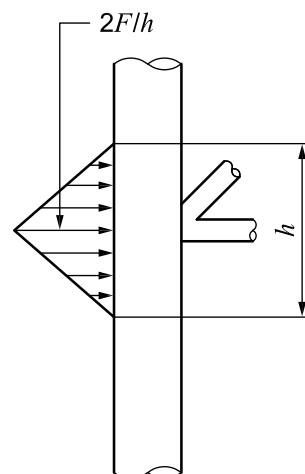


ii) Simplified distribution of the guide reaction

a) Guide mid-bay



i) Guide reaction ignoring guide length



ii) Simplified distribution of the guide reaction

b) Guide at node

Key

- F guide reaction
- h guide length
- s bay height

Figure A.8.5-5 — Correction of point supported guide model for finite guide length

A.8.5.3 Elevating system

A.8.5.3.1 Jacking (or elevating) pinions

The jacking pinions should be modelled using the manufacturer specified pinion stiffness, and should be modelled so that the pinions can resist vertical and the corresponding horizontal forces. A linear spring or cantilever beam can be used to simulate the jacking pinion. The force required to deflect the free end of the cantilever beam a unit distance should be equal to the jacking pinion stiffness. The offset of the pinion/rack contact point from the chord neutral axis should be incorporated in the model.

A.8.5.3.2 Other elevating systems

Elevating system designs not included above should be modelled using stiffness values obtained from the manufacturer/designer, by appropriate system testing or by rational analysis with due consideration of member interface gap spacing and mechanical component stiffness.

A.8.5.4 Fixation system

The fixation system should be modelled to resist both vertical and horizontal forces based on the stiffness of the vertical and horizontal supports and on the relative location of their associated foundations. It is important that the model reflects the local moment strength of the fixation system arising from its finite size and the number and location of the supports.

A.8.5.5 Shock pad – Floating jacking systems

Floating jacking systems generally have two sets of shock pads at each jackcase, one located at the top and the other at the bottom of the jackhouse. Alternatively, shock pads can be provided for each pinion or block of pinions. The jacking system is free to move up or down until it contacts the upper or lower shock pad. In the elevated configuration, the jacking system is in contact with the upper shock pad and in the transit configuration it is in contact with the lower shock pad. The stiffness of the shock pad should be based on the manufacturer's data and the shock pad should be modelled to resist vertical force only. It should also be recognized that the shock pad stiffness characteristics are normally non-linear and can change significantly over time.

A.8.5.6 Jackcase and associated bracing

The stiffness of the jackcase and associated bracing should be modelled accurately since it can have a direct impact on the distribution of horizontal forces between the guides and the jacking system. If the hull is not modelled, it is normally sufficient to restrain the base of the jackcase and associated bracing, as well as the foundations of the fixation system and the lower guide structures at their connections to the hull.

A.8.5.7 Equivalent leg-to-hull stiffness

The determination of stiffnesses for the equivalent leg-to-hull connection model can be accomplished by the following means.

- The application of unit load cases to a detailed leg model in combination with a detailed leg-to-hull connection model in accordance with 8.3.2 and 8.5: Unit load cases are applied as described in A.8.3.3. The effective stiffness of the connection can be determined from the differences between the results from the detailed leg model alone (see A.8.3.3) and those from the detailed leg plus leg-to-hull connection model as follows.
 - Axial unit load case: This case is used to determine the vertical leg-to-hull connection stiffness, K_{vh} from the axial end displacements of the detailed leg model, Δ , and the axial end displacements of the combined leg and leg-to-hull connection model, Δ_C , under the action of the same unit load case, F , as given in Equation (A.8.5-1):

$$K_{vh} = F/(\Delta_C - \Delta) \quad (\text{A.8.5-1})$$

- Pure moment applied either as a moment or as a couple: This case is used to derive the rotational leg-to-hull connection stiffness, K_{rh} from either the end slopes, θ_M and θ_C , or the end deflections, δ and δ_C , of the two models under the action of the same end moment, M , as given in Equation (A.8.2-2):

$$K_{rh} = M/(\theta_C - \theta_M) \text{ or } K_{rh} = ML/(\delta_C - \delta) \quad (\text{A.8.5-2})$$

- Pure shear, which can be used to determine the horizontal leg-to-hull connection stiffness, K_{hh} , in a similar manner, accounting for the rotational stiffness already derived: Normally, the horizontal leg-to-hull connection stiffness can be assumed infinite.

If the model contains non-linearities, e.g. due to the inclusion of gap elements, care should be taken to ensure that suitable magnitudes of unit load cases are applied to accurately linearize the connection stiffness for the final anticipated displacement including wind actions, etc.

A.8.6 Modelling the spudcan and foundation

A.8.6.1 Spudcan structure

When modelling the spudcan, rigid beam elements are considered sufficient to achieve an accurate transfer of the seabed reaction into the leg chords and bracing. It should be noted that, due to the sudden change in stiffness, these rigid beams can cause artificially high stresses at the leg to spudcan connections. Hence, the modelling and selection of element type should be carefully considered when an accurate calculation of leg member stresses is required in this area.

For a strength analysis of the spudcan and its connections to the leg, a detailed model with appropriate boundary conditions should be developed. This analysis can be performed on an independent model of the spudcan.

A.8.6.2 Seabed reaction point

Unless geotechnical analyses demonstrate otherwise, the vertical position of the reaction point at each spudcan should be located at a distance above the spudcan tip equivalent to

- a) half the maximum predicted penetration (when spudcan is partially penetrated); or
- b) half the height of the spudcan (when the spudcan is fully penetrated).

The legs of an independent leg jack-up can be either assumed to be pinned or supported with translational and rotational foundation springs at the reaction point. The assumed boundary conditions should be clearly stated together with the assumptions for any moment fixity provided to the spudcans by the soil.

The spudcan geometry, sloping seabeds, bottom obstructions, existing spudcan 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 that consider the spudcan geometry and seabed topology under the action of preload and should, normally, only be taken into account where this is detrimental to the assessment results. In such cases, the strength of the spudcan should also be considered.

Non-symmetrical geometries should be specially considered.

Further discussion on seabed reaction is contained in Clause 9.

A.8.6.3 Foundation modelling

Methods of establishing the degree of rotational restraint, or fixity, at the spudcans are discussed further in Clause 9 and A.9. Upper or lower 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 seabed reaction forces. This percentage is normally 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 (see 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 and A.8.8.3.

A.8.8 Application of actions

A.8.8.1 Assessment actions

The assessment follows a partial factor format. The partial action factors are applied to the actions defined in other clauses (i.e. they are action factors, not action-effect factors). The jack-up response is non-linear and, hence, the application of the combined factored actions does not in general develop the same result as the factored combination of individual action effects.

The actions and action effects are discussed in turn below.

A.8.8.2 Functional actions due to fixed load and variable load

The actions on the hull due to fixed load and variable load should be applied to the model in such a manner as to represent their correct vertical and horizontal distribution. The hull functional actions are the hull masses multiplied by the vertical gravitational acceleration. The hull mass distribution can be represented by a combination of self-generated mass and applied point masses at the node points of the model. When redistribution of the hull weight is used to correct for hull sag moment (A.8.8.3), the correct horizontal weight distribution can be compromised; when this is undesirable, one of the alternative approaches in A.8.8.3 should be used.

The mass and weight modelling of the legs is more complex than for the hull (see A.8.7). Separate mass and functional action models should consistently account for buoyancy, marine growth, added mass, entrapped water, etc.

In benign areas, the ULS environment is sometimes within the defined SLS limits for the jack-up and the assessment metocean conditions do not exceed the limits for changing to the elevated storm mode (see 5.3). In such cases, the assessment should be for the ULS environment and the proposed operating mode configurations, e.g. with increased variable load, cantilever extended and unequal leg loads. Individual leg reactions under the functional actions can approach the preload reaction. A small additional leg reaction due to environmental actions can then result in additional spudcan penetration.

When the operations manual permits the variable load to be increased as metocean conditions reduce, the jack-up should be assessed to the ULS for operational environments and/or lower return periods (see 5.3). This is of particular importance in areas where significant additional penetrations are possible.

A.8.8.3 Hull sagging

When a jack-up is installed on site, the legs normally engage the seabed with the hull supported by its own buoyancy in a hogged condition. Subsequently, with the hull slightly clear of the water, preload ballast is taken on board thus preloading the legs to achieve their final penetration. This normally leads to an extreme hull sagging condition. Finally, the preload ballast is dumped and the hull elevated to the required elevation for the

site. In this configuration, the hull is sagging under self-weight and variable load. The leg shear and bending moments caused by hull sagging are very dependent on leg guide clearances, the design and operation of the jacking system operational parameters, etc. Such moments should be considered in the assessment analyses, and are larger in shallow waters where the leg extension below the hull is small and consequently the leg bending stiffness is higher.

An FE model with distributed hull stiffness and distributed functional actions incorporates hull sag effects if the functional actions are applied to the jack-up in its initially undeflected shape at the operating hull elevation. The hull sag moment is generally overpredicted by this modelling technique and may be reduced by up to 75 % of the value that would be obtained from an analysis using a hull model with

- a) the maximum extreme storm weight distributed according to A.8.8.2;
- b) guide clearances set to zero; and
- c) the elevating system loads equalized within each leg.

The reduction of the hull sag moment should be achieved by one or more of the following:

- applying correcting moments to the hull in the vicinity of each leg;
- redistributing the hull weight, whilst maintaining the correct centre of gravity;
- including realistic guide clearances; and/or
- adjusting position of the spudcan reaction point (prescribed displacement).

Methods that affect the stiffness of the model such as increasing the hull stiffness or increasing the 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 to the increase in hull weight and the revised distribution, e.g. 25 % of the initial hull sag plus 100 % of the sag due to the change to the operating condition.

A.8.8.4 Metocean actions

A.8.8.4.1 Wind actions

Wind actions are determined from 7.3.4. The wind actions on the legs above and below the hull should be modelled to represent their correct vertical and horizontal distribution. Actions can be applied as distributed or as nodal actions. Where nodal actions are used, a sufficient number should be applied to reflect the distributed nature of the actions, and it should be ensured that the correct total shear and overturning moment are achieved on each leg.

Similarly, the wind actions on the hull and associated structure can be applied as distributed or as nodal actions. The application should also ensure that the correct total shear and overturning moment on the hull are achieved.

A.8.8.4.2 Wave/current actions

Wave/current actions are determined from 7.3.3. The wave/current actions on the leg and the spudcan structures above the sea floor should be modelled to represent their correct vertical and horizontal distribution. Where nodal actions are used, their application should ensure that the correct total shear and overturning moment are achieved on each leg, and reflect the distributed nature of the actions.

A.8.8.5 Inertial actions

A deterministic dynamic storm analysis requires the explicit determination of an inertial loadset, F_{in} (see Clause 10). This loadset should be applied to the model in combination with the other actions.

For the SDOF approach, F_{in} is applied to the hull as lateral force(s) acting through the hull centre of gravity.

When the inertial loadset is derived from a random dynamic analysis, the applied loadset should match both the inertial base shear and the inertial overturning moment. This can be accomplished by a combination of

- a) lateral force(s) acting on the hull;
- b) lateral force(s) acting equally on all the legs above the upper guide in the direction of the metocean actions; and
- c) correcting moment(s) applied as a horizontal or vertical couple(s) to the hull.

The ratio of the total lateral forces acting on the legs above the hull to the lateral forces acting on the hull should not exceed the ratio of the mass of the legs above the upper guide to the total mass of the hull. The moment due to the lateral forces applied to the legs above the upper guide should not exceed the correcting moment required to match the overturning moment, i.e. when applying the forces in b) above, the correcting moment in c) should increase the overturning moment.

Forces or moments due to inertial actions should normally be applied only to structure above the lower guide. Internal leg forces and foundation forces are both important aspects of a site-specific assessment and application of inertial actions to the legs below the lower guide directly affects these in an unrealistic manner.

NOTE The application of the inertial loadset using concentrated forces can result in spurious local stresses.

A.8.8.6 Large displacement effects

There are two displacement effects that it is necessary to capture:

- lateral displacement of the hull causes the functional actions to increase global OTM (global P- Δ effects); and
- Euler amplification of local member forces increases member stresses (local p- δ effects).

The assessor 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 member 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 understand 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 19905-2:2012, A.8.

The correction term is

$$-P_g/L$$

where

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

L is the distance from the spudcan reaction point to the hull vertical centre of gravity.

This negative stiffness correction term applied at the hull produces an additional lateral force at the hull proportional to the structural deflection. The resulting (additional) base overturning moment is equal to P_g times the hull displacement.

The negative stiffness is incorporated into the global stiffness matrix by attaching orthogonal horizontal translational spring elements to a node(s) representing the hull centre of gravity. If sets of orthogonal springs are attached to the hull in the vicinity of each leg, using the total spring stiffness divided by the number of legs, the torsional stiffness is also corrected.

If the negative spring(s) are earthed, the additional lateral force (due to the negative stiffness term) causes an overprediction of the horizontal leg reactions. Typically, this is not critical and the horizontal reactions at each leg can be reduced by an amount equal to the force in the spring divided by the number of legs. However, when non-linear foundation elements are used, the earthed-spring approach overpredicts the horizontal foundation reactions and results in erroneous foundation responses. The overprediction of the horizontal leg reactions can be avoided if sets of negative horizontal springs are defined for each leg and connected between the hull and the spudcan.

The application of negative springs to the model accounts for global displacement effects but does not include local Euler effects for individual members; therefore, code checks should include appropriate terms to account for amplification of local moments (see A.12.4).

A.8.8.7 Conductor actions

The conductor actions can be applied as static forces. The reaction due to the tension and hydrodynamic action on the conductor should be included in the jack-up's global analysis model and applied through the support point on the hull.

The effects of stiffness and damping in the conductor are not generally modelled in a jack-up structural assessment because they normally have negligible influence on the global jack-up response.

Structural integrity assessment of an individual conductor is outside the scope of this part of ISO 19905.

A.8.8.8 Earthquake actions

No guidance is offered.

A.9 Foundations

A.9.1 Applicability

No guidance is offered.

A.9.2 General

No guidance is offered.

A.9.3 Geotechnical analysis of independent leg foundations

A.9.3.1 Foundation modelling and assessment

A.9.3.1.1 General

In 9.3.1 and A.9.3.1 are addressed the approaches to foundation modelling for

- response analysis;
- foundation assessment checks.

The response analysis should incorporate dynamic effects using a compatible or conservative foundation model. Dynamic effects can either be applied by means of a set of added inertial actions or be directly included in the analysis. There is a specific set of foundation assessment checks for each of the foundation models that can be selected for the response analysis, as shown in Table A.9.3-1.

The foundations of independent-leg jack-ups approximate large inverted cones, commonly known as spudcans. Roughly circular in plan, spudcans typically have a shallow conical underside (in the order of 15° to 30° to the horizontal) and can have a sharp protruding point. Other spudcan geometries are not uncommon (see Figure A.9.3-1). Large jack-up spudcans can be in excess of 20 m in diameter, with shapes varying with manufacturer and jack-up. Non-circular spudcans can be approximated by means of a disc with equivalent diameter. The foundation capacity equations given in A.9.3.2 are applicable to circular spudcans. Skin friction on the legs or spudcan is often ignored. Due consideration should be given to the tapered geometry of most spudcans when assessing the foundation capacity.

NOTE Symbols that are not defined in the text can be found in A.4.9.

Dimensions in metres

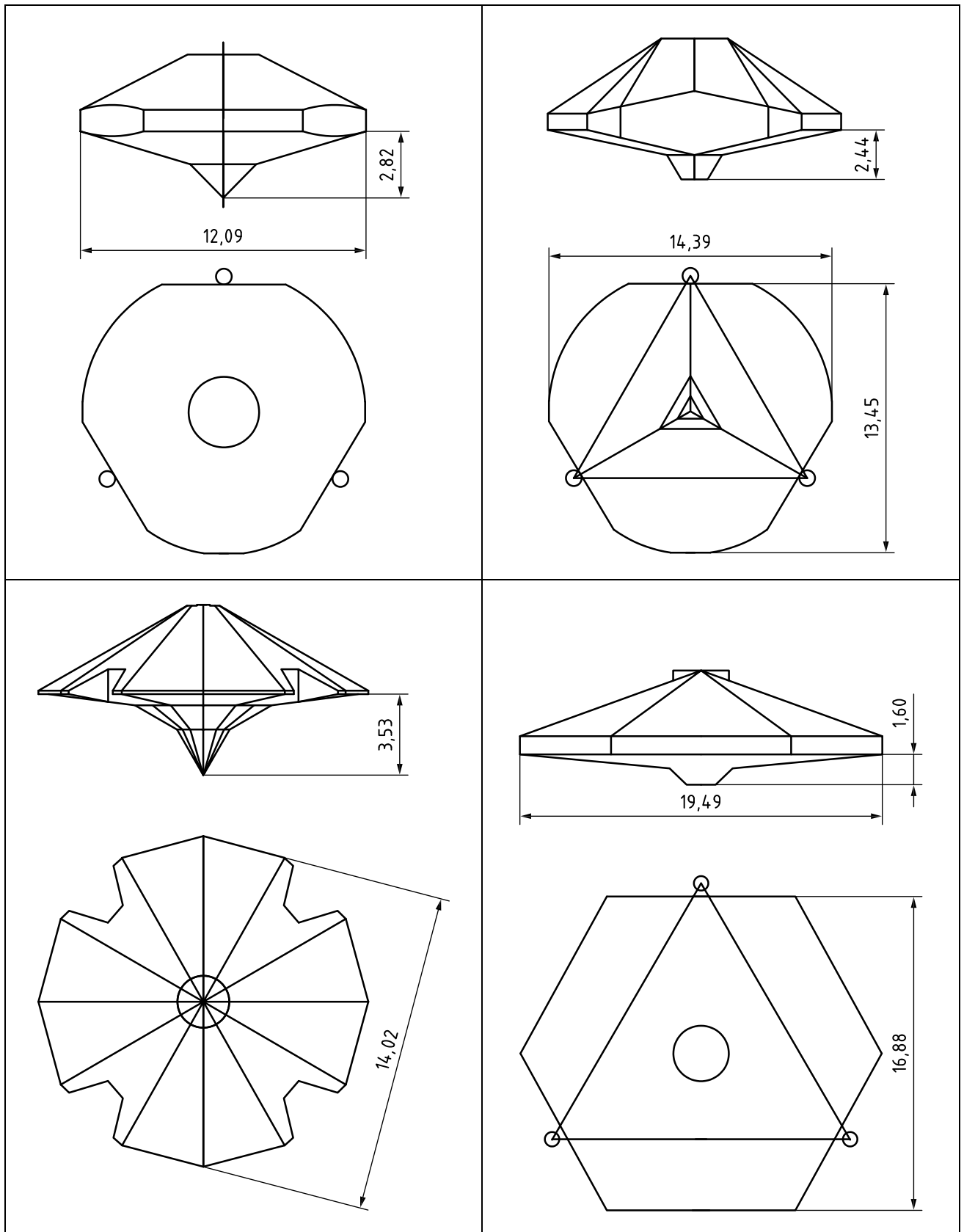


Figure A.9.3-1 — Typical spudcan geometries

A.9.3.1.2 Approaches to foundation assessment

The jack-up and its foundation can be assessed using any of the fixity treatments in Table A.9.3-1. The overall assessment procedure of the jack-up is given in Figure A.10.4-2.

There are certain cases that are not covered in the checks described above, which should be considered separately; some of the more common examples are listed below.

- Cases where the long-term (drained) soil bearing capacity is less than the short-term (undrained) capacity, e.g. for overconsolidated clays or cohesive silts with significant sand seams.
- Cases where a reduction of soil strength occurs due to cyclic loading. This can be of particular significance for silty soils and/or carbonate materials.
- Cases where an increase in spudcan penetration occurs and a potential for punch-through exists, e.g. due to cyclic loading.
- Cases where horizontal seams of weak soil are located beneath the spudcan that can result in inadequate horizontal (sliding) capacity and sliding instability.

If any of the above circumstances exist, further analysis should be carried out.

In the case of partial embedment of a conical spudcan, e.g. in sandy soils, after preloading, additional spudcan embedment can result in a considerable increase in foundation capacity, which can be used in the assessment checks.

Table A.9.3-1 — Approaches to foundation assessment

Fixity treatment in response analysis		Foundation assessment	Acceptance category	Subclause
Pinned		Simple preload check, Windward leg check (both are subject to limitations)	Level 1; Step 1a Level 1; Step 1b	A.9.3.6.2 A.9.3.6.3
		Bearing and sliding checks using vertical-horizontal capacity envelope	Level 2; Step 2a	A.9.3.6.4
		Displacement check using the vertical-horizontal capacity envelope and load-penetration curve; should also meet the Level 2; Step 2a sliding checks	Level 3; Step 3a	A.9.3.6.6
Fixity	Simple interaction surface (secant model)	Bearing and sliding checks (uses the same procedure as in Level 2; Step 2a)	Level 2; Step 2b	A.9.3.6.5
		Displacement check using the vertical-horizontal capacity envelope and load-penetration curve; should also meet the Level 2 sliding checks	Level 3; Step 3a	A.9.3.6.6
	Full interaction surface (yield interaction model)	Foundation checks are implicit in the non-linear model; should also meet the Level 2 sliding checks unless implicitly included	Level 2; Step 2c or Level 3; Step 3b	A.9.3.6.5 A.9.3.6.6
		Continuum	Foundation checks are implicit in the non-linear model	Level 3; Step 3b

A.9.3.1.3 Simple pinned foundation

Pinned foundation treatment incorporates a simple preload and sliding check (both subject to limitations). Otherwise a check on foundation capacity in terms of vertical-horizontal capacity and sliding capacity should be performed.

A.9.3.1.4 Linear vertical, linear horizontal and secant rotational stiffness

This foundation fixity treatment incorporates a check on foundation capacity in terms of vertical-horizontal capacity and sliding capacity. The amount of rotational fixity is not directly involved in a checking equation. However, the moment, bearing and sliding interaction is implicitly checked through the use of the yield surface function. Vertical-horizontal and sliding capacities should still be checked explicitly through the procedures described in A.9.3.6.

A.9.3.1.5 Non-linear vertical, horizontal and rotational stiffness

The vertical, horizontal and moment interaction is implicitly checked through the use of the yield interaction model as described in A.9.3.4.2.4. No other checks are required providing that sliding is incorporated in the model.

A.9.3.1.6 Non-linear continuum foundation model

This model should not be used unless one of the simpler analysis methods above has been used to provide a benchmark for the results. The soil model should be capable of capturing the non-linear behaviour for the strain levels expected in the response. The interface between the spudcan and the soil should be modelled to account for effects such as sliding due to insufficient friction.

A.9.3.2 Leg penetration during preloading

A.9.3.2.1 Analysis method

A.9.3.2.1.1 General

The conventional procedure for the assessment of spudcan load/penetration behaviour is given in the following steps.

- a) Model the spudcan.
- b) Compute the gross ultimate vertical bearing capacity, Q_V , of an open hole for various depths of the bearing area below sea floor using closed form bearing capacity solutions for the best estimate soil strength profile. Lower and upper bound soil strength profiles should also be used to assess the implications of the range of spudcan penetrations.
- c) Use Equations (A.9.3-1) to convert the gross ultimate vertical bearing capacity at each depth to the available structural spudcan reaction, V_L , by deducting, when appropriate, the submerged weight of the backfill, W_{BF} , and adding the soil buoyancy of the spudcan below bearing area, B_S , calculated as $B_S = \gamma V_D$ as described in A.9.3.2.1.5.

$$V_L = Q_V + B_S \quad (\text{with no backfill})$$

$$V_L = Q_V - W_{BF} + B_S \quad (\text{with backfill}) \quad (\text{A.9.3-1})$$

See A.9.3.2.1.4.

- d) Plot the available structural spudcan reaction, V_L as a curve against penetration, accounting for the distance of the spudcan tip beneath the depth of the bearing area by increasing the penetration used in the capacity calculation by this distance. The curve should extend to a suitable depth beyond the expected penetration. This depth should normally be 1,5 times the expected penetration or to the penetration associated with 1,5 times the preload reaction.
- e) Enter the curve of available structural spudcan reaction versus spudcan penetration with the maximum preload reaction at the spudcans and read off the predicted spudcan penetration.

A.9.3.2.1.2 Modelling the spudcan

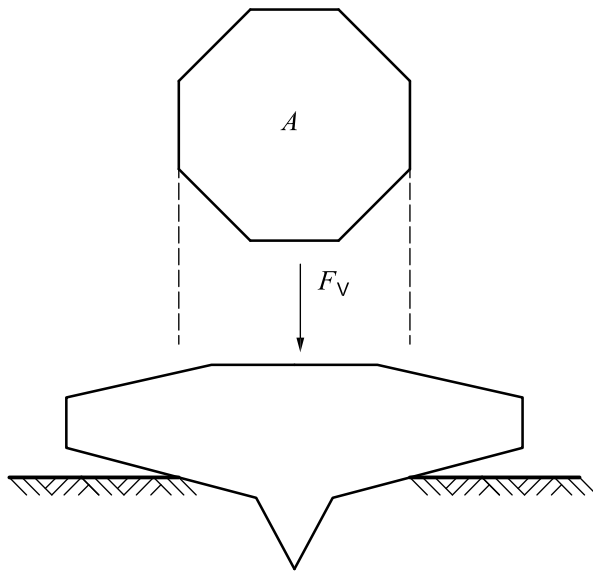
For conventional foundation analyses, the spudcan can often be modelled as a flat circular foundation. The equivalent diameter is determined from the area of the actual spudcan cross-section in contact with the sea floor, or where the spudcan is fully embedded, from the largest cross-sectional area in plan (see Figure A.9.3-2). Foundation analyses are then performed for this circular foundation at the greatest 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.

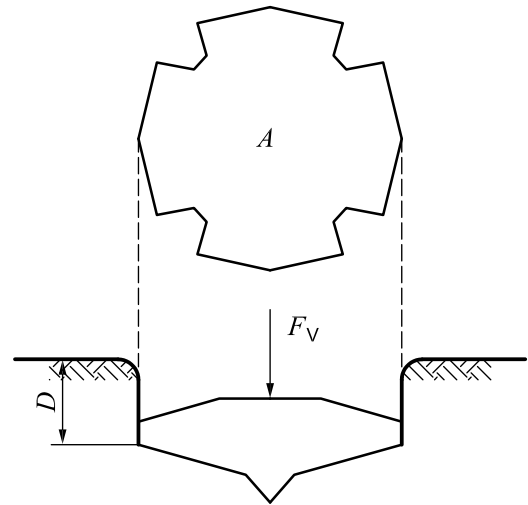
Conical shapes are discussed in Annex E. Other configurations, e.g. rectangular spudcans or legs with significant skin friction, can require alternative treatment.

When a penetration analysis uses bearing capacity factors that account for the conical underside of the spudcan, at each depth the equivalent cone angle (β , Figure A.9.3-3 and Annex E) for the amount of spudcan penetrated should be evaluated. With reference to Figure A.9.3-3, the equivalent cone should be taken such that

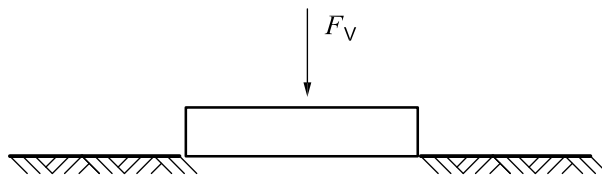
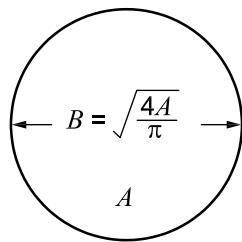
- the diameter, B , of the cone at its top gives an area equal to the largest plan cross-sectional area in contact with the soil;
- the cone angle should be determined so as to enclose the same volume as that of the spudcan below the sea floor; and
- once the largest plan area is mobilized, the volume and equivalent cone angle remain constant.



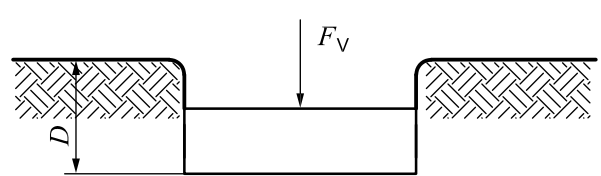
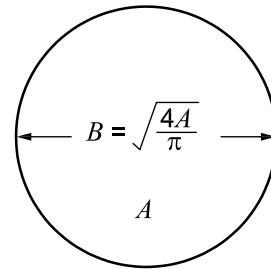
a) Actual spudcan — Partially embedded



b) Actual spudcan — Fully embedded



c) Equivalent model — Partially embedded

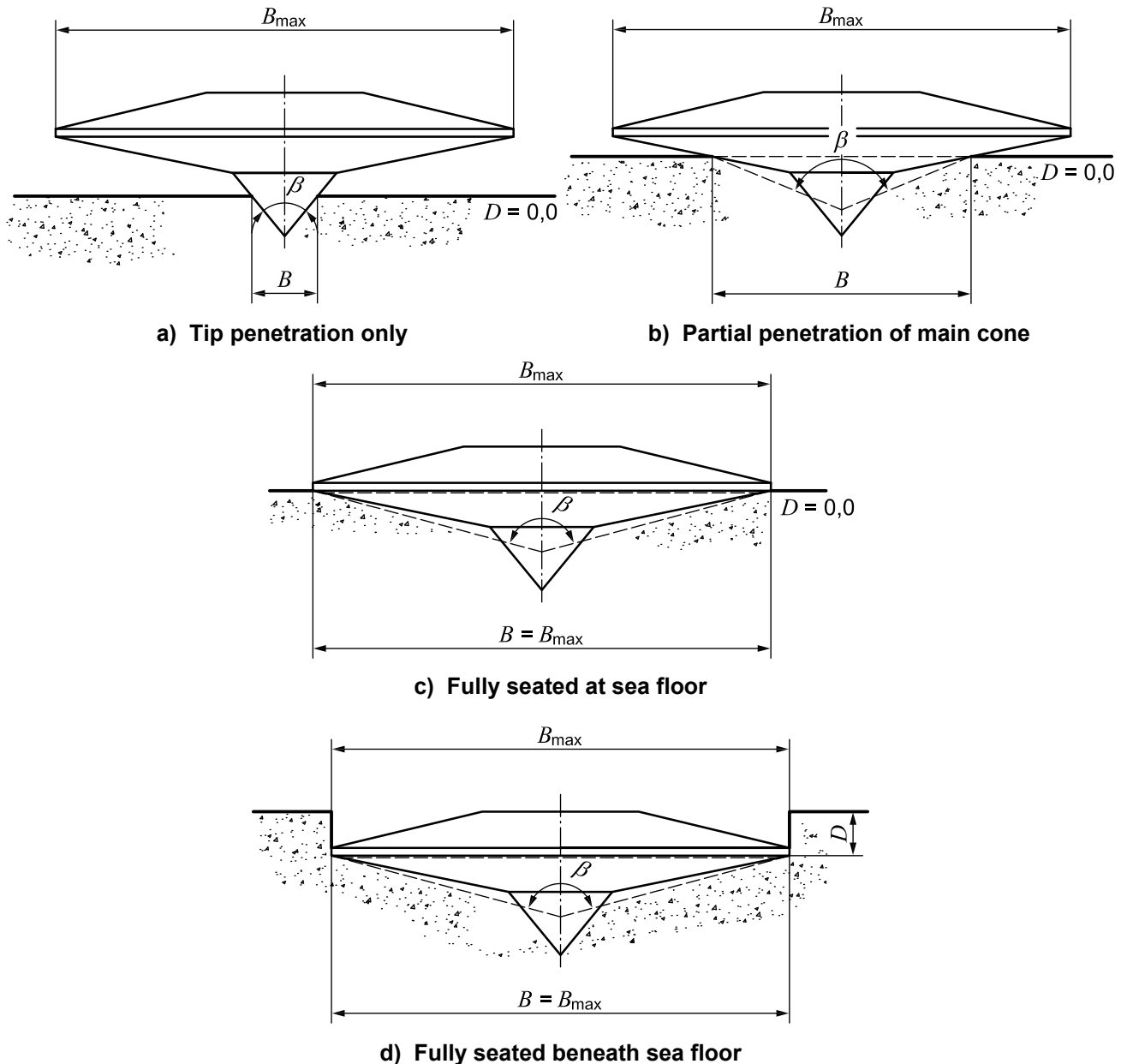


d) Equivalent model — Fully embedded

Key

- A* effective bearing area based on cross-section taken at uppermost part of bearing area in contact with soil
- B* effective spudcan diameter
- D* greatest depth of maximum cross-sectional spudcan bearing area below the sea floor
- F_V gross vertical force acting on the soil beneath the spudcan due to the assessment load case

Figure A.9.3-2 — Spudcan foundation model



Key

- B_{max} maximum effective spudcan diameter
- B effective spudcan diameter
- D greatest depth of maximum cross-sectional spudcan bearing area below the sea floor
- β effective cone angle

NOTE 1 Effective cone indicated by dashed lines.

NOTE 2 Based on Martin, 1994, Reference [A.9.3-1].

Figure A.9.3-3 — Calculating an equivalent conical spudcan for various embedments

A.9.3.2.1.3 Modelling the soil

The soil beneath the spudcan fails as the foundation is loaded during preloading until equilibrium is achieved at the end of the preloading operation. Figure A.9.3-4 shows different failure mechanisms for various soil conditions, which range from conventional bearing capacity failure in uniform soils, potential punch-through for layered soils, squeezing, and combinations of all of these mechanisms. The soil model should be sufficiently accurate to represent the behaviour of spudcan and soil characteristics during preloading.

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 mentioned 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 A.9.3.2.1.4 to A.9.3.2.6.6 address the failure mechanisms shown in Figure A.9.3-4.

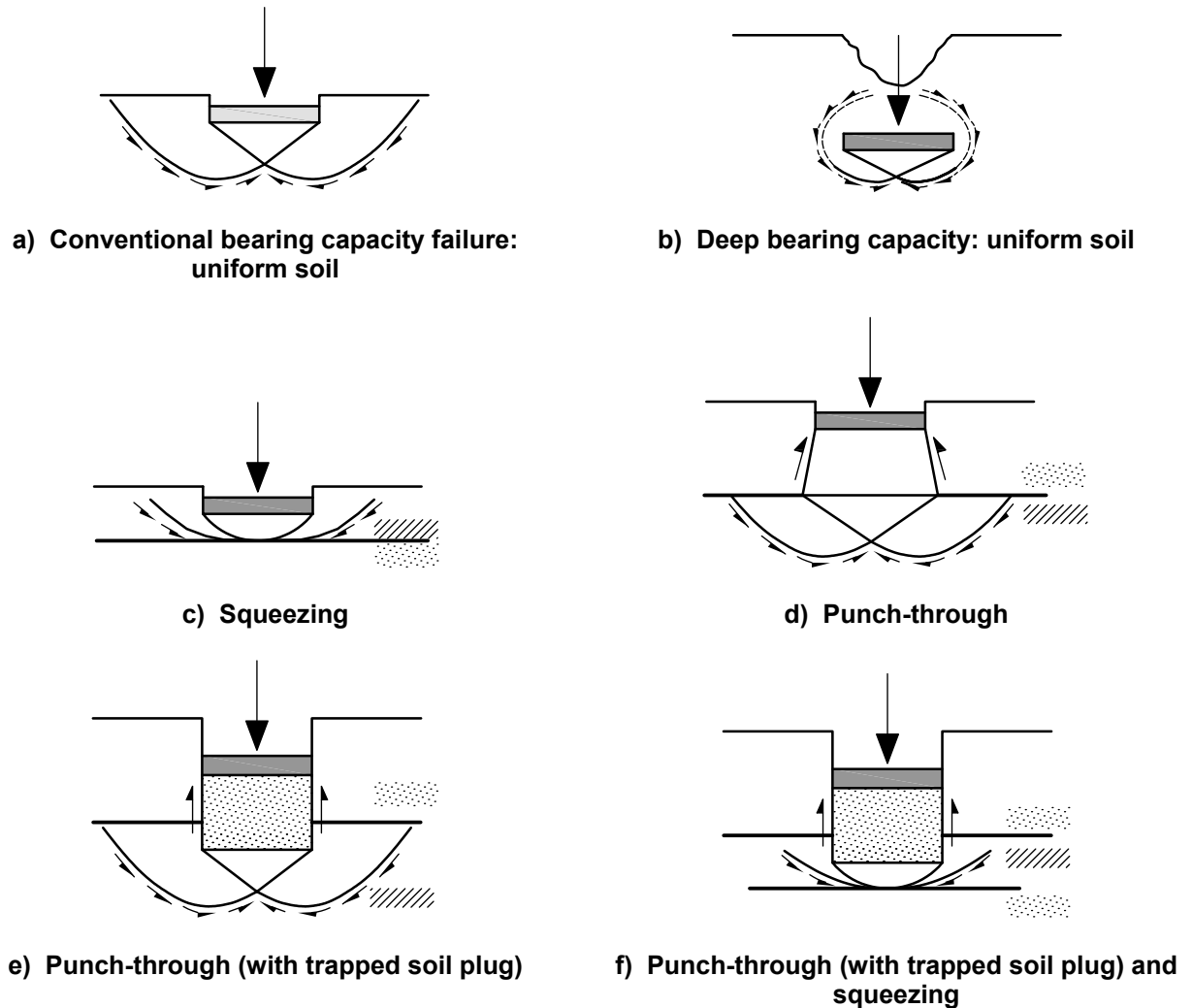


Figure A.9.3-4 — Spudcan bearing failure mechanisms

A.9.3.2.1.4 Backfill

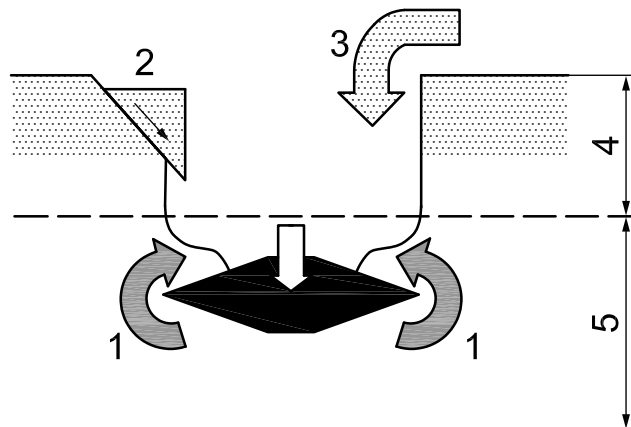
With reference to Figure A.9.3-5, soil backfill on top of the spudcan can result from backflow or infill. Regardless of the mechanism, this soil

- increases penetration if it occurs during preloading;
- reduces capacity available to support downward structural loads at the spudcan if it occurs after preloading;
- always increases the uplift resistance.

Backflow is the soil that flows from beneath the spudcan, around the sides, and onto the top and is more likely to occur in clays than in sands. Backflow can occur at shallow penetrations, but is more likely to occur at

deeper penetrations. In very soft clays, complete backflow is likely to occur. In firm to stiff clays and granular materials, where spudcan penetration is expected to be small, the possibility of backflow diminishes. In general, backflow due to additional penetration during elevated operations is not expected to occur. If it is predicted, the effects should be taken into account.

Infill is the soil on top of the spudcan that results from cavity wall collapse or sediment transport, e.g. where there is a sand veneer over clay. Cavity wall collapse can occur during or after preloading; sediment transport is only of significance after preloading. Cavity wall collapse can occur slowly or suddenly. If it occurs suddenly during preloading, it can cause a rapid increase in penetration.



Key

- 1 backflow
- 2 infill - wall failure
- 3 infill - sediment transport
- 4 region subject to infill processes
- 5 region subject to backflow

NOTE Backfill includes backflow and infill.

Figure A.9.3-5 — Backflow and infill

The submerged weight of backfill ($W_{BF,0}$) during preloading loads the top of the spudcan and results in additional penetration.

Backfill that occurs after preload has been applied and held ($W_{BF,A}$) provides additional weight on the spudcan. This backfill reduces the vertical reaction that the foundation can support to resist the overturning moment. Conversely, any subsequent backfill increases the available uplift capacity of the windward leg(s).

The minimum value of the backfill weight due to backflow during preloading, $W_{BF,0min}$, depends on the limiting depth of cavity, H_{cav} , that remains open above the spudcan during penetration as given in Equations (A.9.3 2):

$$W_{BF,0min} = \gamma[A(D - H_{cav}) - (V_{spud} - V_D)] \quad (\text{with backflow, i.e. } W_{BF,0min} \text{ always positive})$$

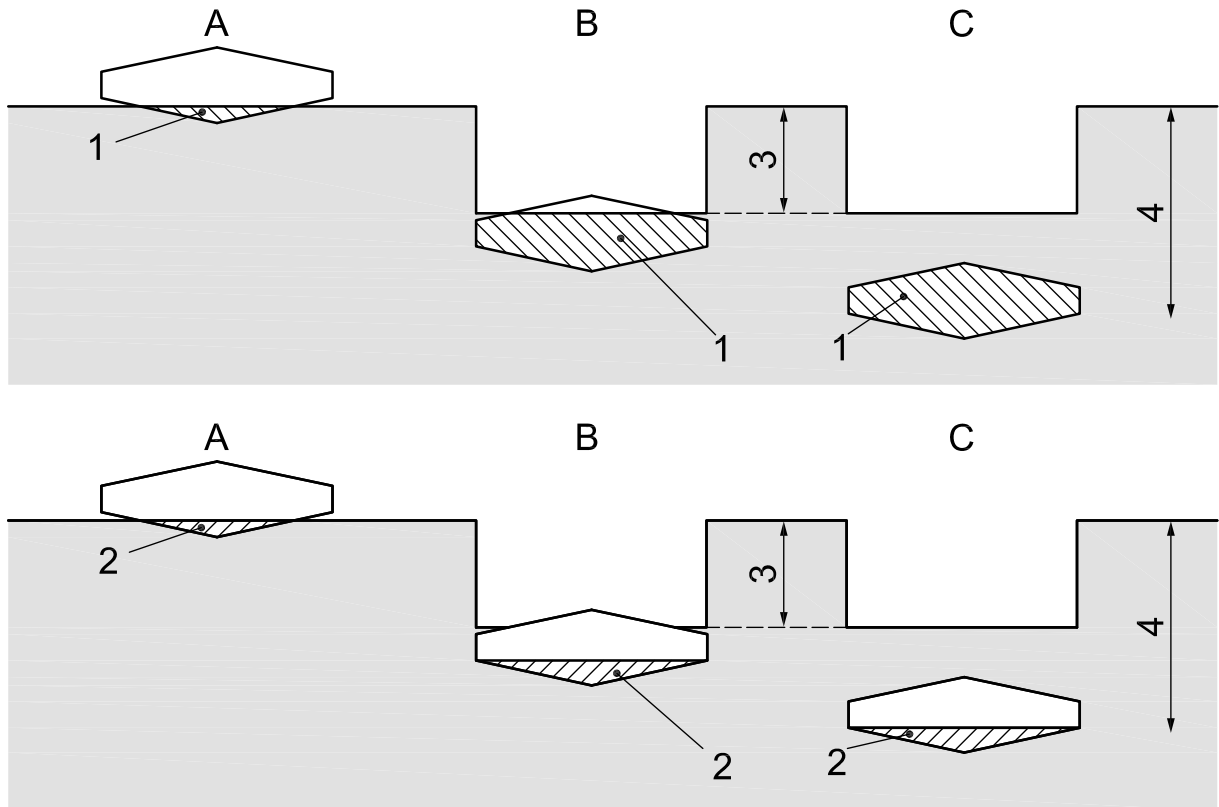
$$W_{BF,0min} = 0 \quad (\text{with no backfill}) \tag{A.9.3-2}$$

where

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

V_D is the volume of the spudcan below the maximum bearing area that is penetrated into the soil, refer to Figure A.9.3-6; V_D is zero for a flat-based spudcan.

Care should be taken when calculating V_{spud} when the spudcan is not fully covered with backflow material; refer to Figure A.9.3-6.

**Key**

- A partial spudcan penetration
- B full spudcan penetration with partial backfill
- C full spudcan penetration with full backfill
- 1 the total volume of the spudcan below the backfill, V_{spud}
- 2 the volume of the spudcan below the maximum bearing area that is penetrated into the soil, V_D
- 3 depth of cavity that remains open above spudcan, H_{cav}
- 4 greatest depth, D , of maximum cross-sectional spudcan bearing area below the sea floor

Figure A.9.3-6 — Definition of spudcan volumes

For a single-layer clay with uniform shear strength or shear strength increasing with depth at a rate, ρ , Equation (A.9.3-3) from Hossain and Randolph^[A.9.3-2] can be used to estimate H_{cav} . This expression and the supporting data are graphically presented in Figure A.9.3-7. Equation (A.9.3-4) can be used to estimate H_{cav} for multi-layer clays with moderate changes of strength, iterating to establish consistent values for H_{cav}/B and s_{uH} .

$$H_{\text{cav}}/B = S^{0,55} - 0,25 S \quad (\text{A.9.3-3})$$

$$H_{\text{cav}}/B = [s_{\text{uH}} / (\gamma' B)]^{0,55} - 0,25 [s_{\text{uH}} / (\gamma' B)] \quad (\text{A.9.3-4})$$

where

$$S = \left(\frac{s_{\text{um}}}{\gamma' B} \right) \left(1 - \frac{\rho}{\gamma'} \right) \quad (\text{A.9.3-5})$$

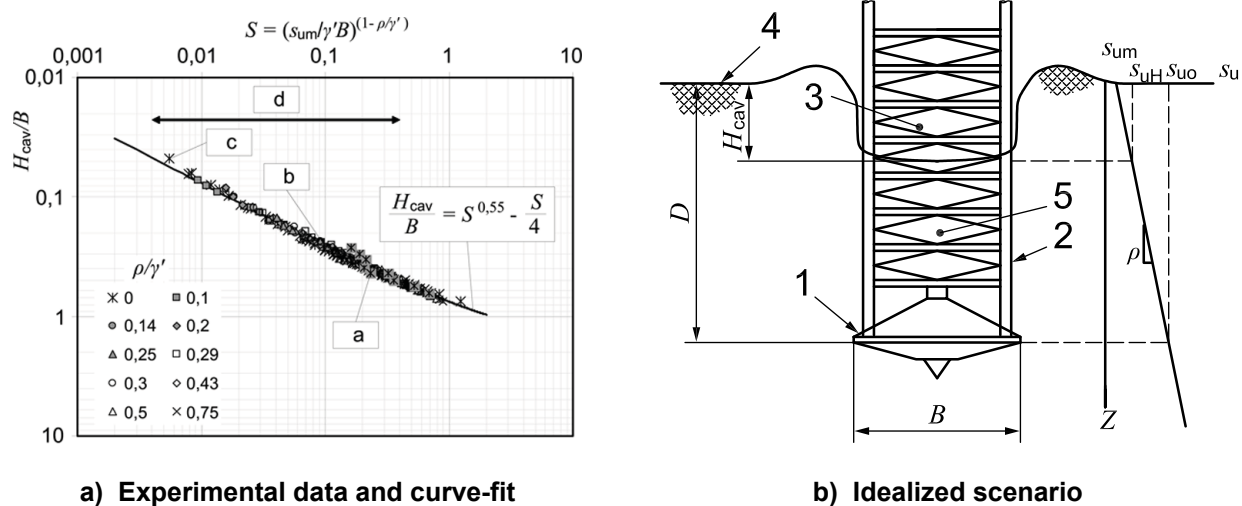
s_{uH} is the undrained shear strength at a depth of H_{cav} below sea floor;

s_{um} is the undrained shear strength at the sea floor.

The onset of backflow marks the transition between shallow and localized failure mechanisms. In the absence of infill, the bearing capacity factor becomes independent of depth for penetrations exceeding the limiting cavity depth, H_{cav} .

In addition to affecting the vertical reaction beneath the spudcan during preloading, the degree of backflow influences the embedment condition of the spudcan and, hence, the uplift resistance (see A.9.4.5), horizontal and moment restraint and, therefore, the yield surface (see A.9.3.3.3).

In silica sand, it is unusual for a conical spudcan to penetrate beyond its widest point. However, if this is predicted, the potential for soil infilling on top of the spudcan should be considered during preloading (as the soil assumes its angle of repose).



Key

- 1 spudcan
- 2 leg truss
- 3 cavity
- 4 mudline
- 5 soil backflow
- B effective spudcan diameter (typically 11 m to 20 m)
- D depth of maximum cross-section in contact with the soil
- H distance from spudcan maximum bearing area to sea floor
- H_{cav} limiting depth of cavity that remains open above the spudcan during penetration
- s_{uH} undrained shear strength at base of cavity
- s_{um} undrained shear strength at sea floor
- s_{u0} undrained shear strength at depth of maximum spudcan bearing area
- s_u undrained shear strength
- Z depth below sea floor
- γ' submerged unit weight of soil
- a Centrifuge test data.
- b Non-uniform strength.
- c Uniform strength.
- d Typical design range.

Figure A.9.3-7 — Estimation of limiting cavity depth, H_{cav} , due to backflow during installation

A.9.3.2.1.5 Required bearing capacity

At maximum preload, the initial gross ultimate bearing capacity, Q_{V0} , under the spudcan is equal to the preload reaction, V_{L0} (see 3.48), plus the submerged weight of any backfill onto the spudcan, less the soil buoyancy of the spudcan below the bearing area as given in Equation (A.9.3-6):

$$Q_{V0} = V_{L0} + W_{BF,o} - B_S \quad (\text{A.9.3-6})$$

where

$W_{BF,o}$ is the submerged weight of the backfill during preloading, which is not less than $W_{BF,omin}$;

$B_S = \gamma' V_D$ is the soil buoyancy of spudcan below bearing area, i.e. the submerged weight of soil displaced by the spudcan below D , the greatest depth of maximum cross-sectional spudcan bearing area below the sea floor;

V_D is the volume of the spudcan below the lowest level of maximum bearing area that is penetrated into the soil; V_D is zero for a flat-based spudcan.

The initial gross ultimate vertical bearing capacity, Q_{V0} , is established by preload operations and related to V_{L0} . However, in some cases, subsequent actions can cause further penetration and a corresponding increase in Q_V , as is consistent with the load-penetration equations given in A.9.3.2.2 through A.9.3.2.6.

A.9.3.2.2 Penetration in clays

The gross ultimate vertical bearing capacity of a foundation in clay of uniform shear strength (undrained failure in clay, $\phi = 0^\circ$) at a specific depth can be expressed as given in Equation (A.9.3-7):

$$Q_V = (s_u \cdot N_c \cdot s_c \cdot d_c + p'_o) \pi B^2 / 4 \quad (\text{A.9.3-7})$$

where

p'_o is the effective overburden pressure at depth, D , of maximum bearing area;

d_c is the bearing capacity depth factor, $d_c = 1 + 0,2 (D/B) \leq 1,5$.

For circular footings, the product $N_c \cdot s_c$ should be taken as 6,0.

For the selection of the design undrained shear strength s_u , an evaluation should be made of the sampling method, the laboratory test type and the field experience regarding the prediction and observations of spudcan penetrations.

Traditionally, the value of N_c has been determined from solutions for strip footing on homogeneous clay, with shape and depth factors based on Skempton^[A.9.3-3]. However, these factors are significantly affected by the gradient of shear strength with depth (see Young *et al.*^[A.9.3-4] and Houlsby and Martin^[A.9.3-5]).

Theoretical solutions for circular conical foundations on clays of uniform and increasing strength with depth have been provided by Houlsby and Martin^[A.9.3-5], as presented in E.1. The solutions give a theoretical lower bound to the soil resistance and should, therefore, provide an upper bound prediction of penetration.

The total bearing capacity factors for rough spudcans, modelled as rough circular plates, are given in Table A.9.3-2 and are valid for the following parameter ranges (see Figures A.9.3-2, A.9.3-3 and A.9.3-7):

- cone angles β between 60° and a flat plate of 180° ;
- embedment depths, D , between 0 and 2,5 diameters;
- values of shear strength gradient $\rho B / s_{um}$ between 0 and 5, where ρ is the rate of increase in undrained shear strength with depth, from a value of s_{um} at the sea floor.

NOTE 1 For soil layers that do not extend to the sea floor surface, s_{um} refers to the undrained shear strength at the top of the layer.

The tables in Annex E provide a theoretical lower bound to the total bearing factor $N_c \cdot s_c \cdot d_c$ to apply to the shear strength at the spudcan base level, s_{u0} , for the full range of the above parameters. Alternatively, Houlsby and Martin^[A.9.3-5] indicates that using the shear strength, s_u , at a depth of $0,09B$ below the spudcan base level together with the bearing factors given in Table A.9.3-2 for a foundation on uniform strength clay provides answers that are within $\pm 12\%$ of the theoretical lower bound solutions.

Alternatively, field experience in the Gulf of Mexico^[A.9.3-4] indicates that for typical Gulf of Mexico shear strength gradients and spudcan dimensions, spudcan penetrations in clay are well predicted by selecting s_u as the average over a depth of $B/2$ below the widest cross-section in combination with the use bearing capacity and depth factors from Skempton^[A.9.3-3].

For clay layers with distinct strength differences, methods for layered soils should be used; see A.9.3.2.6.

Table A.9.3-2 — Bearing capacity factors for rough circular plate on homogeneous clay^[A.9.3-3]

Embedment ratio, D/B	Bearing factor, $N_c \cdot s_c \cdot d_c$
0	6,0
0,1	6,3
0,25	6,6
0,5	7,1
1,0	7,7
$\geq 2,5$	9,0

NOTE 2 The bearing factor is nonlinear with respect to the embedment ratio. It is necessary to use caution when estimating an appropriate bearing factor for embedment ratios other than those given in Table A.9.3-2.

A.9.3.2.3 Penetration in soils with partial drainage

It is recommended that analyses for drained conditions (modelled as sand) and undrained conditions (modelled as clay) be performed to estimate the range of penetrations. Cyclic loading can significantly affect the bearing capacity of silts.

Penetration in soils with partial drainage can be assessed using the approaches described by Finnie and Randolph^[A.9.3-6] and Erbrich^[A.9.3-7].

A.9.3.2.4 Penetration in silica sands

Spudcan penetration in silica sand is usually analysed as a drained process, in which no excess pore water pressure is generated. In drained conditions, the gross ultimate vertical bearing capacity of a circular foundation in homogeneous frictional material can be expressed as given in Equation (A.9.3-8):

$$Q_V = \gamma' d_\gamma N_\gamma \pi B^3 / 8 + p'_o \cdot d_q N_q \pi \cdot B^2 / 4 \tag{A.9.3-8}$$

where

- d_γ is the depth factor on surcharge for drained soils, $d_\gamma = 1,0$;
- d_q is the depth factor for drained soils, $d_q = 1 + 2 \tan \phi' (1 - \sin \phi')^2 \tan^{-1}(D/B)$;
- B is the effective spudcan diameter in contact with the soil;

γ' is the submerged unit weight of the soil;

N_γ and N_q are dimensionless bearing capacity factors calculated for the axisymmetric case (no further shape factor should be applied).

If the spudcan penetrates beyond its widest point, the overburden of soil above this point creates an effective surcharge, p_o' , at the level of the widest point, which leads to additional bearing capacity.

Theoretical values of N_γ and N_q calculated using the slip-line method for a flat, rough circular footing in Martin^[A.9.3-8] are given in Table A.9.3-3 for soil friction angles from 20° to 40°. These N_γ and N_q factors can also be applied to (blunt) conical spudcans that are not fully rough, since the error involved is generally small compared with that arising from the uncertainty in selecting the soil friction angle; for example, Table A.9.3-3 shows that a 1° change in ϕ' gives at least a 20 % change in N_γ . A more detailed penetration analysis can be performed using the values of N_γ for conical footings tabulated in Annex E; these cover a range of cone apex angles and interface roughness coefficients.

Adequate consideration should be given to the selection of an appropriate soil friction angle (see E.2).

Table A.9.3-3 — Bearing capacity factors for a flat, rough circular footing (Martin^[A.9.3-8])

Friction angle ϕ' degrees	Bearing factor N_γ	Bearing factor N_q
20	2,4	9,6
21	2,9	10,9
22	3,5	12,4
23	4,2	14,1
24	5,1	16,1
25	6,1	18,4
26	7,3	21,1
27	8,8	24,2
28	10,6	27,9
29	12,8	32,2
30	15,5	37,2
31	18,8	43,2
32	22,9	50,3
33	27,9	58,7
34	34,1	68,7
35	41,9	80,8
36	51,6	95,4
37	63,7	113,0
38	79,1	134,4
39	98,7	160,5
40	123,7	192,7

A.9.3.2.5 Penetration in carbonate sands

A.9.3.2.5.1 General

Penetrations in carbonate sands are highly unpredictable and can be minimal in strongly cemented materials, or large, in uncemented materials. Cementation, crushable particles, high *in-situ* void ratios and compressibility are some of the characteristics of calcareous sediment that have led to the conclusion that the routine bearing capacity methods linked to the frictional soil strength are inappropriate (Poulos and Chua^[A.9.3-9], Le Tirant and Nauroy^[A.9.3-10] and Finnie and Randolph^[A.9.3-11]). Extreme care should be exercised when operating in these materials.

A.9.3.2.5.2 Uncemented carbonate materials

Relatively large spudcan penetrations have been reported for uncemented carbonate materials despite high laboratory friction angles (Dutt and Ingram^[A.9.3-12]). This can be attributed to either the high compressibility of these materials or low shear strengths due to high voids ratio and a collapsible structure.

The leg penetration is governed by both the strength and deformation characteristics of the soils. The compressibility of carbonate sands is relatively higher than that of silica sands. Hence, greater penetrations should be expected for carbonate sands relative to silica sands despite the similar or even higher laboratory friction angles. This is supported by both experimental studies (Poulos and Chua^[A.9.3-9], Pan^[A.9.3-13], Pan *et al.*^[A.9.3-14], and Byrne and Houlsby^[A.9.3-15]) and theoretical studies (Yeung and Carter^[A.9.3-16]) 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^[A.9.3-9], Houlsby *et al.*^[A.9.3-17], Sharp and van Seters^[A.9.3-18], and Randolph and Erbrich^[A.9.3-19]). 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.9.3-19] explain this bi-linear shape as being attributable to the very small settlement expected before the yield pressure is exceeded.

A.9.3.2.5.4 Predictive methods

The predictions of spudcan penetrations in carbonate sands are likely to be less accurate than those for silica sands because carbonate sands generally have high porosity and a varying degree of cementation.

Spudcan penetration occurs due to a combination of soil compression and soil failure. The use of the conventional general shear failure model for sand for predicting the penetration is, therefore, not appropriate. This model is, however, generally adopted for penetration predictions in carbonate sands but requires a careful assessment of the design friction angle. The reduction of the friction angles is typically in the range of 3° to 7° for cemented and uncemented carbonate sands.

Special attention is required for sites with a surface crust of cemented soil overlying weak, uncemented layers with careful consideration given to the type of punch-through mechanism.

Randolph *et al.*^[A.9.3-22] and Finnie and Randolph^[A.9.3-11] outline a bearing modulus method for uncemented calcareous sands. This is based on the results of a series of centrifuge experiments of model footings that indicate that the vertical bearing capacity increased linearly with depth. An estimation of the bearing pressure can be performed as a function of the overburden pressure rather than the self-weight as given in Equation (A.9.3-9):

$$q_u = \gamma' z N_q \tag{A.9.3-9}$$

where z is the penetration and N_q is the bearing capacity factor. Whilst $N_q \approx 50$ was found to provide reasonable predictions of the centrifuge test data, it can overpredict the foundation bearing capacity of

spudcans in uncemented carbonate soils. Equation (A.9.3-9) can be adapted to calculate the vertical bearing capacity for a conical spudcan by sub-dividing the spudcan geometry vertically into a number of equivalent circular footings as shown in Figure A.9.3-8. The bearing capacity of the area at the base of each slice in contact with the soil can be summed to calculate iteratively the overall bearing capacity of the conical footing for different footing penetrations.

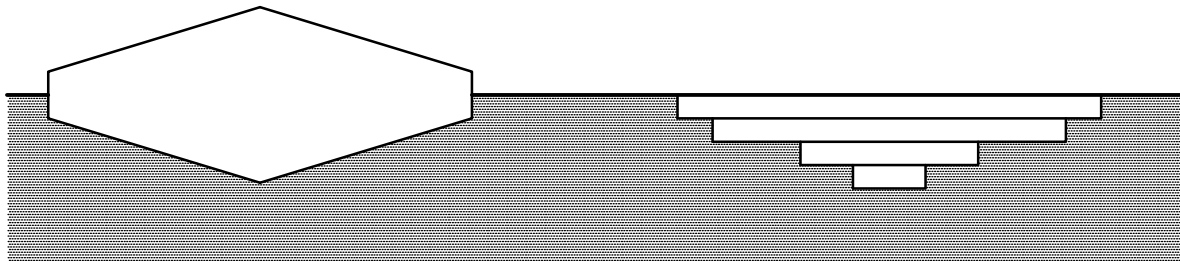


Figure A.9.3-8 — Representation of a conical spudcan by equivalent circular footing “slices” for the calculation of vertical bearing capacity in carbonate sands

Other predictive methods for circular spudcans on both cemented and uncemented calcareous sands have been published, including Islam^[A.9.3-20], Islam *et al.*^[A.9.3-21], Houlsby *et al.*^[A.9.3-17], Randolph *et al.*^[A.9.3-22], Finnie and Randolph^[A.9.3-11], and Yamamoto *et al.*^{[A.9.3-23],[A.9.3-24]}. 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.9.3-23],[A.9.3-24]} provide simple equations for the response of shallow footings on compressible sands.

A.9.3.2.6 Penetration in layered soils

A.9.3.2.6.1 General

Three different foundation failure mechanisms should be considered when making spudcan predictions in layered soils:

- a) general shear;
- b) squeezing;
- c) punch-through.

The first failure mechanism occurs if soil strengths of subsequent layers do not vary significantly. Thus, an average soil strength (either s_u or ϕ) can be determined below the spudcan. The spudcan penetration versus foundation capacity relationship is then generated using criteria from A.9.3.2.2 to A.9.3.2.5.

Criteria for the other two failure mechanisms (squeezing and punch-through) are given in A.9.3.2.6.2 to A.9.3.2.6.6. Punch-through is of particular significance since it concerns a potentially dangerous situation where a strong layer overlies a weak layer and, hence, a small additional spudcan penetration can be associated with a significant reduction in vertical bearing capacity that results in rapid leg penetration.

Backflow and infill should be considered.

A.9.3.2.6.2 Squeezing of clay

On a soft clay subject to squeezing overlying a significantly stronger layer (see Figure A.9.3-9), the gross ultimate vertical bearing capacity of a spudcan can be analysed by methods given by Brown and Meyerhof^[A.9.3-25] and by Vesic^[A.9.3-26] in combination with the bearing capacity and depth factors given by Skempton^[A.9.3-3] as given in Equation (A.9.3-10).

$$Q_v = A \left\{ \left(a_s + \frac{b_s B}{T} + \frac{1,2D}{B} \right) s_u + p'_o \right\} \geq A \{ N_c s_c d_c s_u + p'_o \} \tag{A.9.3-10}$$

where

$$d_c = 1 + 0,2 D/B$$

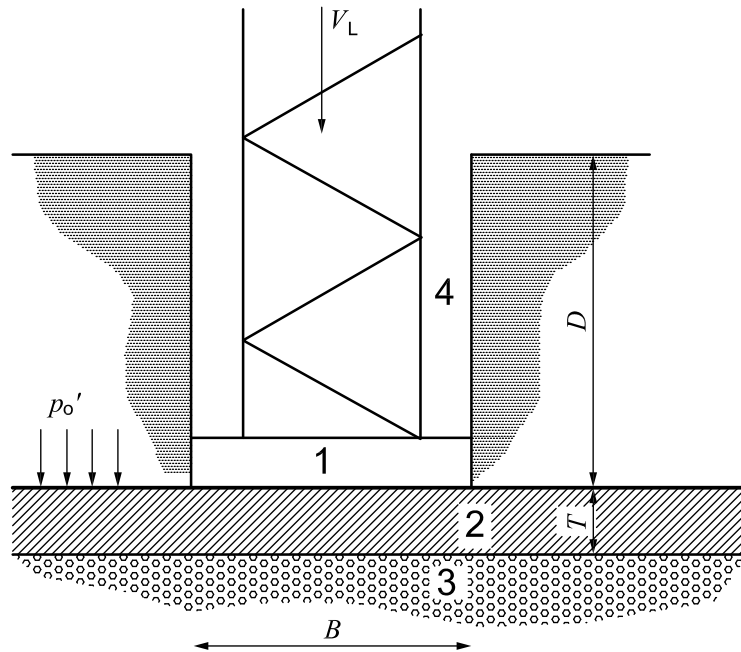
and the following squeezing factor constants are recommended:

$$a_s = 5,00$$

$$b_s = 0,33$$

and s_u is the undrained shear strength of the soft clay layer.

It is pointed out that the lower bound vertical foundation capacity is given by general failure in the clay layer [right hand side of Equation (A.9.3-10)], and that squeezing occurs when $B \geq 3,45T (1 + 1,025D/B)$ for $D/B \leq 2,5$. The upper bound capacity (for $T \ll B$) is determined by the ultimate bearing capacity of the underlying strong soil layer.



Key

- 1 spudcan with effective bearing area, A
- 2 softer clay layer with shear strength, s_u
- 3 stronger soil
- 4 no backflow and no infill (i.e., no backfill)
- B effective spudcan diameter
- D depth of spudcan below sea floor
- V_L available spudcan reaction; see Equation (A.9.3-1)
- p'_o effective overburden pressure at depth, D
- T thickness of weaker clay layer beneath the spudcan

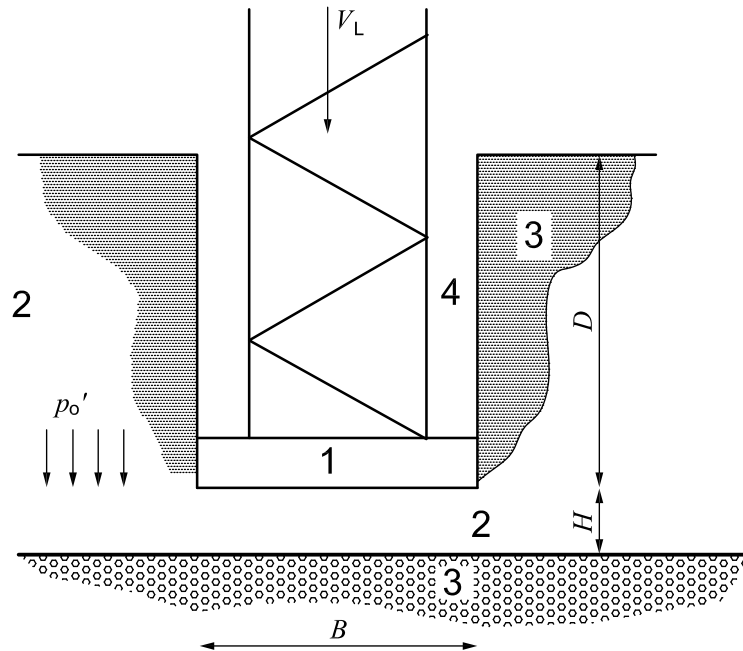
Figure A.9.3-9 — Spudcan bearing capacity analysis — Squeezing clay layer

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.9.3-25] as given in Equation (A.9.3-11); (see Figure A.9.3-10):

$$Q_V = A \left(3 \frac{H}{B} s_{u,t} + N_c s_c s_{u,b} \right) \leq A N_c s_c s_{u,t} \quad (\text{A.9.3-11})$$

Equation (A.9.3-11) applies to clay layers of uniform undrained shear strengths.



Key

- 1 spudcan with effective bearing area, A
- 2 stronger clay layer with shear strength, $s_{u,t}$
- 3 weaker clay layer with shear strength, $s_{u,b}$
- 4 no backflow and no infill (i.e., no backfill)
- B effective spudcan diameter
- D depth of spudcan below sea floor
- V_L available spudcan reaction; see Equation (A.9.3-1)
- p_o' effective overburden pressure at depth, D
- H distance from spudcan to weaker layer below

Figure A.9.3-10 — Spudcan bearing capacity analysis — Two clay layers

A.9.3.2.6.4 Punch-through — Sand overlying clay

The gross ultimate vertical bearing capacity of a spudcan on a sand layer overlying a weak clay layer can be computed using a load spread model (see Figure A.9.3-11). In this model, the bearing capacity of the spudcan, Q_V , is calculated by considering a fictitious footing at the interface between the sand and clay layers. Be aware that this is a convenient method for expressing the bearing capacity of the spudcan within the layered soil profile and is not a representation of the actual “punching shear” failure mechanism.

The fictitious footing has an equivalent diameter as given in Equation (A.9.3-12):

$$B' = B + 2Hn_s \quad (\text{A.9.3-12})$$

For sand overlying clay, a load spread factor, n_s , of 3 (see Figure A.9.3-11) has been recommended by Young and Focht^[A.9.3-27] for jack-up foundations. However, comparison with model test data (Jacobsen *et al.*^[A.9.3-28], Higham^[A.9.3-29], and Craig and Chua^[A.9.3-30]) suggests a range of n_s from 3 to 5. Conversely, actual spudcan penetration data are available that suggest smaller n_s values (Baglioni^[A.9.3-31]). It is, therefore, recommended that load spread factors in the range of 3 to 5 be used, consistent with current industry practice.

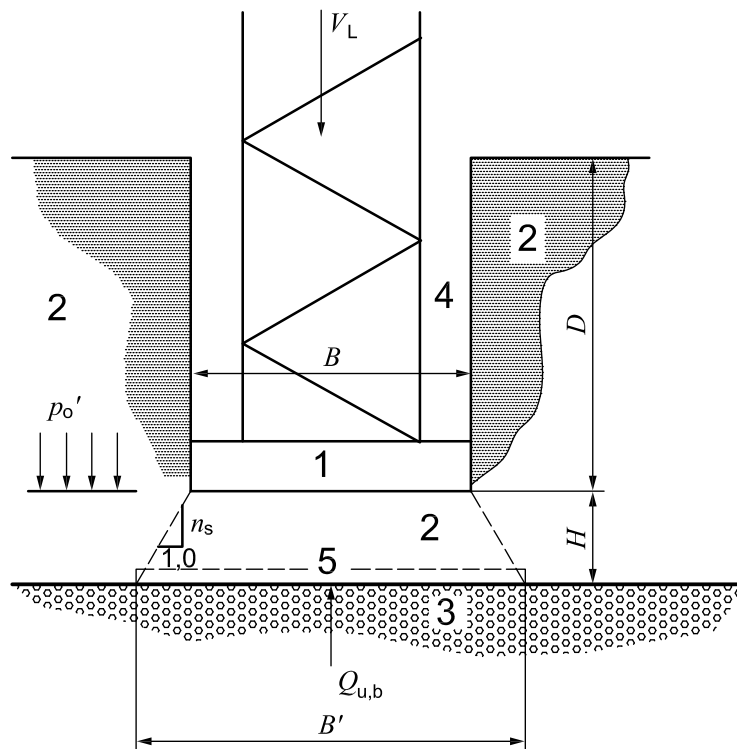
The calculation of the bearing capacity of the fictitious footing should include consideration of the weight of the sand, W , between the base of the actual spudcan and the fictitious footing at the surface of the lower (clay) layer as given in Equation (A.9.3-13):

$$W = 0,25 \pi (B + 2H/n_s)^2 H \gamma' \tag{A.9.3-13}$$

The total capacity is, therefore, as given in Equation (A.9.3-14):

$$Q_v = Q_{u,b} - W \tag{A.9.3-14}$$

where $Q_{u,b}$ is the ultimate vertical foundation bearing capacity for the fictitious footing at the interface between the sand and clay layers with no backfill, which can be calculated using Equation (A.9.3-7).



Key

- 1 spudcan with effective diameter, B
- 2 sand layer with submerged unit weight of γ'
- 3 clay layer
- 4 void above spudcan, i.e. no backflow and no infill (i.e., no backfill)
- 5 fictitious spudcan with effective diameter, B' , at the interface between the upper and lower layers
- D depth of actual spudcan below the sea floor
- V_L available spudcan reaction; see Equation (A.9.3-1)
- H distance from spudcan to clay layer below
- n_s load spread factor for sand overlying clay (typically 3 to 5)
- p_o' effective overburden pressure at depth D

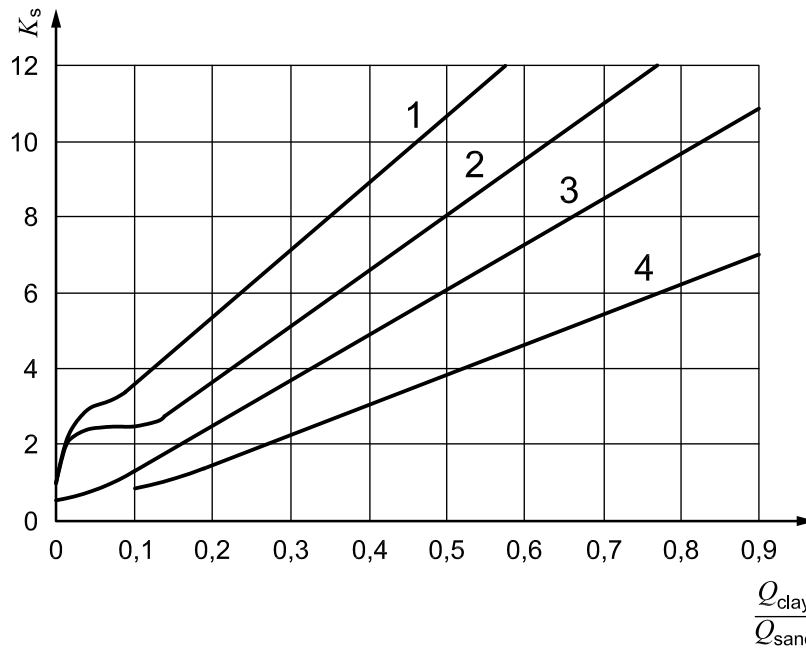
Figure A.9.3-11 — Spudcan bearing capacity analysis — Sand over clay

Alternatively, the gross ultimate initial bearing capacity may be calculated using Equation (A.9.3-15) derived from Hanna and Meyerhof^[A.9.3-32]:

$$Q_v = Q_{u,b} - AH \gamma' + 2AH(H\gamma' + 2p'_o)K_s \tan \phi B \tag{A.9.3-15}$$

where $Q_{u,b}$ is determined according to A.9.3.2.2, assuming that the spudcan bears on the surface of the lower clay layer with no backfill.

The punching shear coefficient, K_s , depends on the strength of both the sand layer and the clay layer, which can be derived from the graphs in the reference paper, Hanna and Meyerhof^[A.9.3-32]; see Figure A.9.3-12.



Key

- 1 $\phi' = 40^\circ$
- 2 $\phi' = 35^\circ$
- 3 $\phi' = 30^\circ$
- 4 $\phi' = 25^\circ$
- K_s coefficient of punching shear
- Q_{clay} bearing capacity of clay
- Q_{sand} bearing capacity of sand
- ϕ' effective angle of internal friction for sand in degrees

Figure A.9.3-12 — Bearing capacity ratio versus coefficient of punching shear for spudcans

A new approach based on a centrifuge study has been proposed by Teh *et al.*^[A.9.3-33]. The load-penetration curve typical of the punch-through condition is represented by a simplified profile consisting of three characteristic bearing capacities, namely bearing capacity at sea floor, Q_0 (at $d = 0$), maximum bearing capacity, Q_{peak} (at $d = d_{crit}$), and bearing capacity in the underlying clay (for $d \geq H$). A brief description of the approach is provided in E.3.

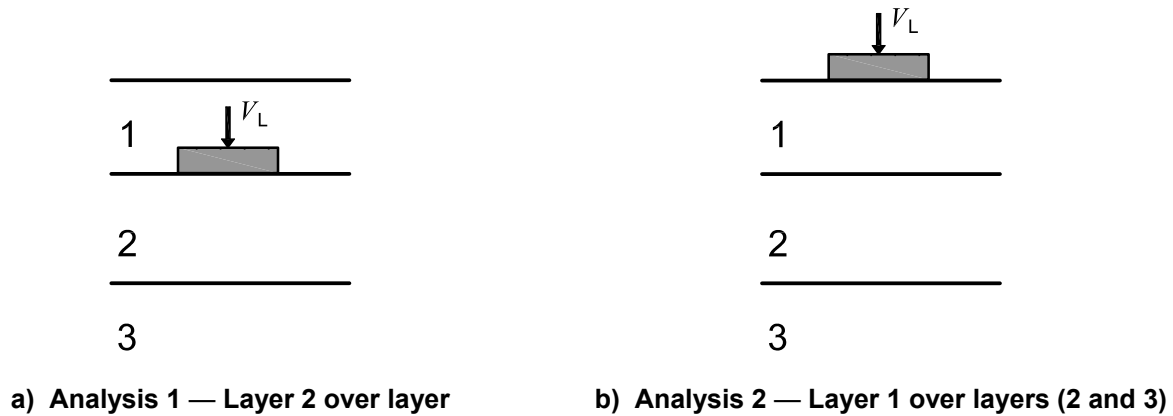
A.9.3.2.6.5 Punch-through — Cemented crust over weak soil

The occurrence of a cemented crust overlying a weak layer of clay or loose sand/silt should be carefully considered. The analysis relies on accurate information on the thickness and strength of the crust and the strength of the underlying layer. The analysis can be performed using simplified load spread models or

advanced numerical models. The potential for punch-through can be significantly affected by the shape of the spudcan and its tip.

A.9.3.2.6.6 Three layered systems

The gross ultimate vertical bearing capacity of a spudcan at the top of a three soil layer system can be computed using the squeezing and punch-through criteria for two layer systems. Firstly, the bearing capacity of a spudcan with diameter B at the top of the lower two layers (layers 2 and 3 in Figure A.9.3-13) is computed. These two layers can then be treated as one (lower) layer in a subsequent two layer system analysis involving the upper layer (layer 1 in Figure A.9.3-13). Analysis for the top layer can incorporate load spread effects.



Key

- 1 layer 1
- 2 layer 2
- 3 layer 3
- V_L available spudcan reaction see Equation (A.9.3-1)

Figure A.9.3-13 — Spudcan bearing capacity analysis — Three-layer case

A.9.3.3 Yield interaction

A.9.3.3.1 General

During preloading, the soil beneath the spudcan fails plastically and the spudcan penetrates until the bearing capacity is in equilibrium with the preload reaction. When the preload is removed, the soil unloads on the small strain unload-reload stiffness curve. The spudcan geometry and the soil properties at the penetrated position are then used to determine the maximum moment and horizontal capacities that, with the vertical capacity, are the principal values that define the size of the yield interaction surface.

The limiting combinations of the spudcan moment, vertical and horizontal reactions are defined by the yield interaction surface; see Figure A.9.3-14. Inside the yield surface the foundation behaviour is considered to be elastic for small strains, but it becomes increasingly inelastic as the yield surface is approached. On the yield surface, the foundation undergoes inelastic deformation with increased reaction beneath the spudcan. Provided the jack-up's preload capacity is appropriate for a site's environmental conditions, the majority of the foundation load-deflection behaviour during a storm should be essentially elastic and only a few, if any, extreme events cause stiffness reduction.

When the foundation is considered as pinned, the yield surface degenerates to a vertical-horizontal load space.

A.9.3.3.2 to A.9.3.3.6 are applicable to traditional spudcan designs. Guidance for the foundation behaviour of spudcans fitted with skirts is provided in A.9.4.1.

The modelling approach to the interaction of vertical, horizontal and rotational forces on the spudcan was initially developed for shallow foundations based on a plasticity relationship; see Dean *et al.*^[A.9.3-34], Cassidy *et al.*^[A.9.3-35], Wong and Murff^[A.9.3-36], Baerheim^[A.9.3-37] and Van Langen and Hoppers^[A.9.3-38]. The plasticity relationship can account for moment softening at high loading levels, unloading behaviour and work-hardening effects. The shape of the yield surface for shallow foundations is paraboloidal.

In clay, a deeply embedded spudcan can achieve a greater moment capacity than a spudcan with a shallow penetration (see Templeton *et al.* ^[A.9.3-39] ^[A.9.3-40] ^[A.9.3-41]). In addition, the shape of the yield surface changes from paraboloidal to becoming progressively more ellipsoidal with increasing penetration. This was first shown experimentally by Martin and Houlsby^[A.9.3-42], further substantiated via numerical analysis by Martin and Houlsby^[A.9.3-43] and confirmed via finite element analysis by Templeton *et al.*^[A.9.3-40]. This effect can be taken into account by interpolating between the paraboloidal shape of the shallow embedment yield surface [obtained by setting $a = 0$ in Equation (A.9.3-16)] and the ellipsoidal shape for deep embedments ($D > 2,5B$) using the depth interpolation parameter, a . Accomplishment of the necessary interpolation via a single parameter linear variation of the coefficients was shown to be sufficiently accurate by Templeton^[A.9.3-41].

This model does not include sliding; where sliding is important, this should be incorporated separately using the method described in A.9.3.5.

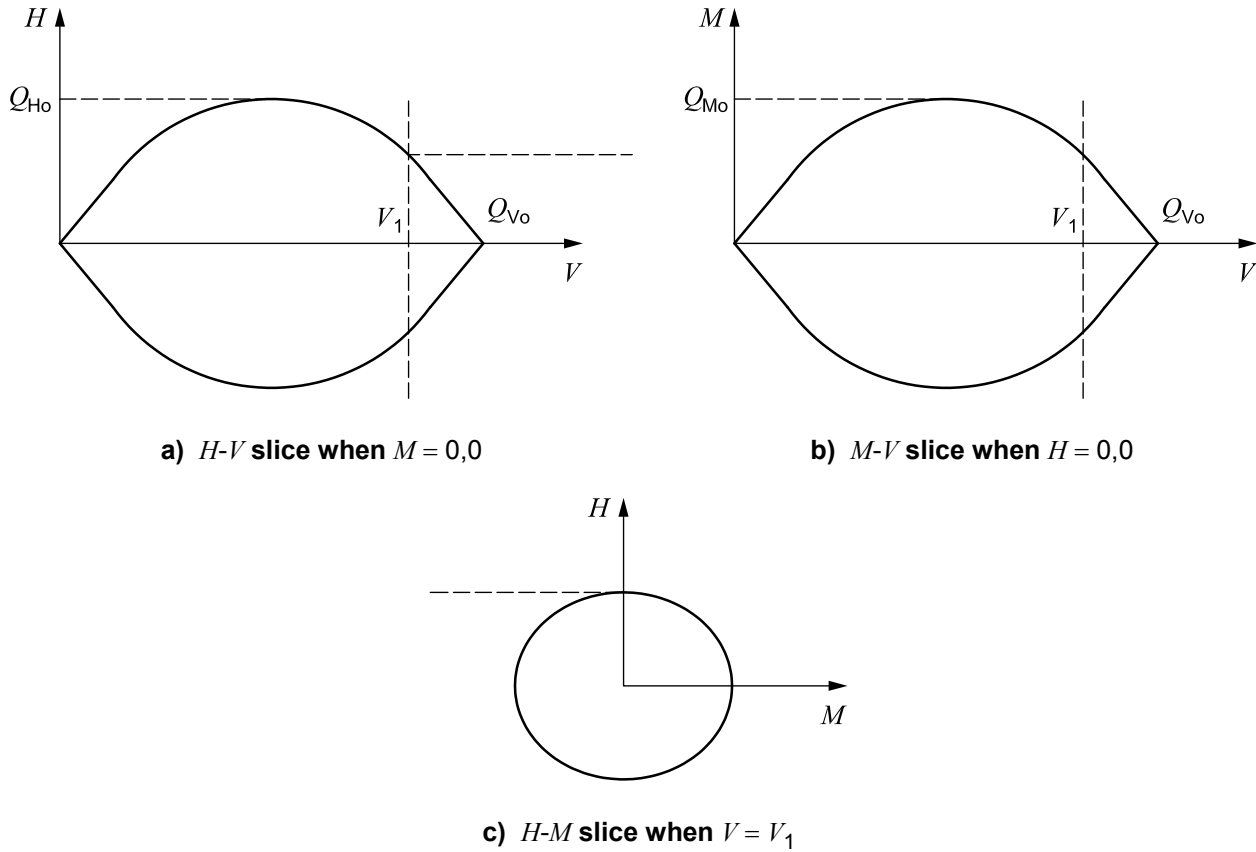
There is no existing data for deeply embedded spudcans in sand. The application of the yield surface calibrated to shallow penetrations is likely to be conservative for the deep penetration case.

In the yield equation, the gross ultimate vertical bearing capacity, Q_V , is initially established by preload operations and related to V_{L0} as specified by Equation (A.9.3-6). However, in some cases, subsequent environmental actions can cause further penetration and a corresponding increase in Q_V , as is consistent with the load-penetration equations given in A.9.3.2.2 through A.9.3.2.6. In assessment analyses that incorporate work hardening, such possible increases in Q_V can be included automatically. In other types of analyses, the effects of such increases in Q_V can be included via calculations using the load-penetration equations, together with values of any additional penetration. In either case, care should be taken to include all contributions from P- Δ effects associated with leaning due to the additional penetration. Consideration should be given to the possibility of excess penetration, rapid penetration and/or punch-through.

The forces F_H and F_V and the moment F_M acting on the spudcan are the forces transferred to the foundation by the jack-up in operational, extreme storm or earthquake conditions due to the assessment load case F_d in 8.8. They include quasi-static contributions due to factored actions, and contributions from dynamic response, as appropriate, in accordance with the procedures of Clause 10.

- F_H is the horizontal force applied to the spudcan due to the assessment load case F_d (see 8.8).
- F_V is the gross vertical force acting on the soil beneath the spudcan due to the assessment load case F_d (see 8.8).
- F_M is the moment applied to the spudcan due to the assessment load case F_d (see 8.8).

If a force combination (F_V, F_H, F_M) satisfies Equation (A.9.3-16) for the interaction yield surface, then this combination lies on the yield surface. The force combination (F_V, F_H, F_M) lies outside the yield surface if the left-hand side of Equation (A.9.3-16) is greater than zero. Conversely, the force combination lies inside the yield surface if the left-hand side is less than zero.



Key

- H* horizontal capacity
- M* moment capacity
- V* vertical capacity

Figure A.9.3-14 — Three slices through the three-dimensional yield surface (at $M = 0,0$; $H = 0,0$; and $V = V_1$ constant)

A.9.3.3.2 Ultimate vertical/horizontal/rotational capacity interaction function for spudcans in sand and clay

The general equation, Equation (A.9.3-16), from Templeton^[A.9.3-41] can be used for fully or partially penetrated spudcans:

$$\left[\frac{F_H}{Q_H} \right]^2 + \left[\frac{F_M}{Q_M} \right]^2 - 16(1-a) \left[\frac{F_V}{Q_V} \right]^2 \left[1 - \frac{F_V}{Q_V} \right]^2 - 4a \left[\frac{F_V}{Q_V} \right] \left[1 - \frac{F_V}{Q_V} \right] = 0 \tag{A.9.3-16}$$

where, for the vertical direction:

Q_V is the gross ultimate vertical bearing capacity of the soil beneath the spudcan. In the absence of additional penetration $Q_V = Q_{Vo}$, the capacity achieved during preloading, as defined in A.9.3.2.1.5;

F_V is the gross vertical force acting on the soil beneath the spudcan due to the assessment load case, F_d (see 8.8) as given in Equations (A.9.3-17):

$$F_V = V_{st} - B_S \tag{with no backfill}$$

$$F_V = V_{st} + W_{BF,o} + W_{BF,A} - B_S \tag{with backfill} \tag{A.9.3-17}$$

V_{st} is the vertical force applied to the spudcan due to the assessment load case, F_d (see 8.8), which includes quasi-static contributions due to factored actions and contributions from dynamic response, as appropriate, in accordance with the procedures of Clause 10, and also includes leg weight and water buoyancy but excludes the submerged weight of backfill ($W_{BF,0} + W_{BF,A}$) and spudcan soil buoyancy (B_S);

where, for the horizontal direction and moment,

F_H is the horizontal force applied to the spudcan due to the assessment load case, F_d (see 8.8);

F_M is the bending moment applied to the spudcan due to the assessment load case, F_d (see 8.8).

a) The clay formulation is given in Equations (A.9.3-18) to (A.9.3-25) [variables for sand can be found in b)].

$$\begin{aligned} Q_H &= C_H(Q_V - p'_o \pi B^2/4) \\ &= C_H Q_{Vnet} \end{aligned} \quad \begin{array}{l} \text{(see Notes 1 and 2)} \\ \text{(A.9.3-18)} \end{array}$$

$$\begin{aligned} Q_M &= [0,1 + 0,05a(1+b/2)] (Q_V - p'_o \pi B^2/4) B \\ &= [0,1 + 0,05a(1+b/2)] Q_{Vnet} B \end{aligned} \quad \begin{array}{l} \text{(see Note 1)} \\ \text{(A.9.3-19)} \end{array}$$

$$\begin{aligned} a &= D/2,5B && \text{for } D < 2,5B && \text{(see Note 3)} \\ &= 1,0 && \text{for } D \geq 2,5B && \text{(see Note 3)} \end{aligned} \quad \begin{array}{l} \text{(A.9.3-20)} \\ \text{(A.9.3-20)} \end{array}$$

where

p'_o is the effective overburden pressure at depth, D , of maximum spudcan bearing area;

$$b = (D_b s_{u,a}) / (D s_{u0}) \quad \begin{array}{l} \text{(see Note 4)} \\ \text{(A.9.3-21)} \end{array}$$

$$Q_{Vnet} = (s_u \cdot N_c \cdot s_c \cdot d_c) \pi B^2/4 \quad \text{(A.9.3-22)}$$

$$\begin{aligned} C_H &= C_{Hshallow} + (C_{Hdeep} - C_{Hshallow}) D/B && \text{for } D < B && \text{(see Note 4)} \\ &= C_{Hdeep} && \text{for } D \geq B && \text{(see Note 4)} \end{aligned} \quad \begin{array}{l} \text{(A.9.3-23)} \\ \text{(A.9.3-23)} \end{array}$$

where

D_b is the depth of backflow (see A.9.3.2.1.4), equal to $(D - H_{cav})$; infill should not be considered;

s_u is the undisturbed undrained shear strength;

$s_{u,a}$ is the undrained shear strength of backfill material above the spudcan, accounting for disturbance and soil sensitivity;

$s_{u,l}$ is the undisturbed undrained shear strength at the spudcan tip;

s_{u0} is the undisturbed undrained shear strength at deepest depth of maximum bearing area (D below sea floor);

$$C_{Hshallow} = [s_{u0}^A + (s_{u0} + s_{u,l}) A_s] / Q_{Vnet} \quad \text{(A.9.3-24)}$$

$$C_{Hdeep} = [1,0 + (s_{u,a}/s_{uo})] [0,11 + 0,39(A_s/A)] \quad (A.9.3-25)$$

NOTE The formulation given in Equation (A.9.3-25) for the case of deep embeddings in clay is partly based on the finite element results in Templeton^[A.9.3-44], and reduces to Equation 2 in that paper for the case of $s_{u,a} = s_{uo}$.

where

A is the spudcan effective bearing area based on cross-section taken at uppermost part of bearing area in contact with soil (see Figure A.9.3-2);

A_s is the spudcan laterally projected embedded area (the projection of the area in contact with the soil).

b) The sand formulation is given in Equations (A.9.3-26) to (A.9.3-27).

$$\begin{aligned} Q_H &= 0,12 (Q_V - p'_o \pi B^2/4) \\ &= 0,12 Q_{Vnet} \quad (\text{see Note 1}) \quad (A.9.3-26) \end{aligned}$$

$$\begin{aligned} Q_M &= 0,075 B (Q_V - p'_o \pi B^2/4) \\ &= 0,075 B Q_{Vnet} \quad (\text{see Note 1}) \quad (A.9.3-27) \end{aligned}$$

$$a = 0,0$$

where

p'_o is the effective overburden pressure at depth, D , of maximum spudcan bearing area;

$$Q_{Vnet} = (\gamma' d_\gamma N_\gamma \pi B^3/8) + (p'_o d_q N_q \pi B^2/4) - (p'_o \pi B^2/4);$$

d_γ is the depth factor on surcharge for drained soils; $d_\gamma = 1,0$;

B is the maximum effective spudcan diameter in contact with the soil;

γ' is the submerged unit weight of the soil;

N_γ is a dimensionless bearing capacity factor calculated for the axisymmetric case (no further shape factor should be applied).

For sand, the values of $0,12Q_{Vnet}$ and $0,075BQ_{Vnet}$ are based on experimental evidence that includes Tan^[A.9.3-45], Gottardi and Butterfield^{[A.9.3-46][A.9.3-47]}, Gottardi *et al.* ^[A.9.3-48], Byrne and Houlsby^[A.9.3-15], Bienen *et al.* ^[A.9.3-49], and Cassidy^[A.9.3-50]. There are no existing data for spudcans deeply embedded in sand. The application of these parameters, which are calibrated to shallow penetrations, is likely to be conservative for the deep penetration case.

At zero vertical 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 $F_V < 0,5 Q_V$. In such cases, the yield surface expansion given in A.9.3.3.3 may be used. For deep penetration cases where suction capacity exists, Q_V can be less than zero and the yield surface may be enlarged; the simplified expansion given in A.9.3.3.3 should not be used.

NOTE 1 The moment capacities are calculated as a function of the product of the net vertical bearing capacity and the effective spudcan diameter. The horizontal capacity in sand or clay is calculated as a function of the net vertical bearing

capacity. For clay, the net vertical bearing capacity is used because the weight of soil on top of the spudcan does not affect the horizontal and moment capacities. For sand, the use of net capacity is conservative because it neglects the increase in capacity due to the weight of any soil on top of the spudcan which has a beneficial effect on the horizontal and moment capacities. For the case of shallow embedment in clay, a conservative value for C_H can be established by considering minimal embedment of a flat-bottomed spudcan on very strong clay where the horizontal capacity per unit base area is given by the shear strength, and the vertical capacity per unit base area is approximately six times the shear strength, so that: $Q_H = 0,16 Q_{Vnet}$. This value can be used as an alternative, conservative, horizontal capacity expression for shallow embedment in clay.

NOTE 2 According to Andersen^[A.9.3-56], for clays susceptible to cyclic degradation (i.e. with OCR, $R_{OC} \geq 4$), cyclic degradation reduces the horizontal capacity by 30 %, i.e. the horizontal capacity calculated from static soil properties should be multiplied by a reduction factor of 0,7.

NOTE 3 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 $a = 0$ can be used.

NOTE 4 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.

In many cases, simpler forms of the yield interaction equation can be used. Results from finite element analysis (see Templeton *et al.*^[A.9.3-40] or Templeton^[A.9.3-41]) indicate that insignificant error is incurred by the use of the value, $a = 0$ for embedment less than $0,3B$ or by the use of the value, $a = 1$ for embedment greater than $1,7B$.

In the case of $a = 0$, the yield interaction equation reduces to the paraboloidal form given in Equation (A.9.3-28):

$$\left(\frac{F_H}{Q_H}\right)^2 + \left(\frac{F_M}{Q_M}\right)^2 - 16\left(\frac{F_V}{Q_V}\right)^2 \left(1 - \frac{F_V}{Q_V}\right)^2 = 0 \quad (\text{A.9.3-28})$$

In the case of $a = 1$, the yield interaction equation reduces to the fully ellipsoidal form given in Equation (A.9.3-29):

$$\left(\frac{F_H}{Q_H}\right)^2 + \left(\frac{F_M}{Q_M}\right)^2 - 4\left(\frac{F_V}{Q_V}\right) \left(1 - \frac{F_V}{Q_V}\right) = 0 \quad (\text{A.9.3-29})$$

Equation (A.9.3-16) for the yield surface can be conveniently rewritten to give the maximum available moment on the spudcan F_M as a function of the applied horizontal and vertical forces as given in Equation (A.9.3-30):

$$F_M = Q_M \left[16(1-a) \left(\frac{F_V}{Q_V}\right)^2 \left(1 - \frac{F_V}{Q_V}\right)^2 - \left(\frac{F_H}{Q_H}\right)^2 + 4a \left(\frac{F_V}{Q_V}\right) \left(1 - \frac{F_V}{Q_V}\right) \right]^{0.5} \quad (\text{A.9.3-30})$$

This equation only applies when

$$0 < F_V < Q_V$$

and the condition given in Equation (A.9.3-31) is satisfied:

$$0 < 16(1-a) \left(\frac{F_V}{Q_V}\right)^2 \left(1 - \frac{F_V}{Q_V}\right)^2 - \left(\frac{F_H}{Q_H}\right)^2 + 4a \left(\frac{F_V}{Q_V}\right) \left(1 - \frac{F_V}{Q_V}\right) \quad (\text{A.9.3-31})$$

A.9.3.3.3 Spudcans in clay with $F_V < 0,5 Q_V$

The yield surface in the region $0 < F_V/Q_V < 0,5$ (typically applicable to windward legs) can be replaced by an adhesion envelope that provides additional horizontal and moment capacity due to spudcan-soil adhesion. The adhesion envelope is applicable for vertical load levels less than $(F_V/Q_V)_t$ which defines the tangent intercept between the adhesion envelope and the standard form of the yield surface and is dependent upon the adhesion factor, α , and the “a” parameter that defines the form of the yield surface. The adhesion envelope can be expressed as given in Equation (A.9.3-32):

$$\left(\frac{F_H}{f_1 Q_H}\right)^2 + \left(\frac{F_M}{f_2 Q_M}\right)^2 - 1,0 = 0 \tag{A.9.3-32}$$

where

$$f_1 = \alpha + m_\alpha \left(\frac{F_V}{Q_V}\right) \tag{A.9.3-33}$$

$$f_2 = f_1 \text{ where suction (i.e. uplift resistance) is available, or} \tag{A.9.3-34}$$

$$f_2 = \sqrt{16(1-a) \left(\frac{F_V}{Q_V}\right)^2 \left(1 - \frac{F_V}{Q_V}\right)^2 + 4a \left(\frac{F_V}{Q_V}\right) \left(1 - \frac{F_V}{Q_V}\right)} \text{ where suction cannot be relied upon;}$$

$$\alpha = 1,0 \text{ for soft clays } (s_u = 20 \text{ to } 40 \text{ kPa}), \text{ or} \tag{A.9.3-35}$$

$$\alpha = 0,5 \text{ for stiff clays } (s_u = 75 \text{ kPa to } 150 \text{ kPa}), \text{ or}$$

α is determined by linear interpolation when $40 < s_u < 75$;

m_α is the gradient of the adhesion envelope.

Figure A.9.3-15 provides a graphical representation of the adhesion envelope and the definitions of the parameters m_α and $(F_V/Q_V)_t$.

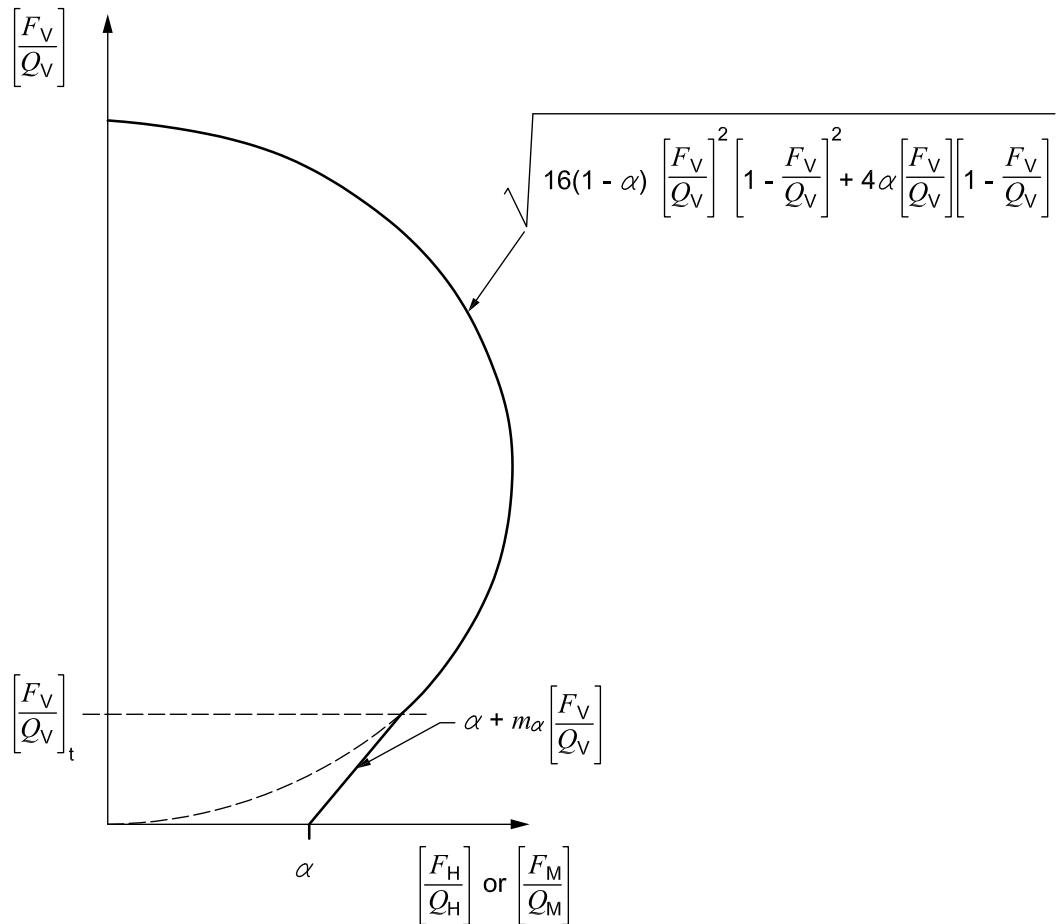


Figure A.9.3-15 — Illustration of the adhesion envelope modification

to the standard yield surface for $F_V < \left(\frac{F_V}{Q_V}\right)_t$

α is the adhesion factor and accounts for the degree of adhesion. The assessor should consider α values within the range of 0,5 to 1,0 depending on site-specific soil data, spudcan/soil interface roughness, etc. When hard clay is present at the surface with an α value below 0,5, the standard form of the yield surface should be used [Equation (A.9.3-16)].

The values for m_α and $\left(\frac{F_V}{Q_V}\right)_t$ have been determined for $a = 0,0$ (paraboloidal) as given in Equations (A.9.3-36) and (A.9.3-37) and for $a = 1,0$ (ellipsoidal) as given in Equations (A.9.3-38) and (A.9.3-39):

— For $a = 0$:

$$m_\alpha = 4(1 - \sqrt{\alpha}) \quad (\text{A.9.3-36})$$

$$\left(\frac{F_V}{Q_V}\right)_t = \sqrt{\frac{\alpha}{4}} \quad (\text{A.9.3-37})$$

— For $a = 1$:

$$m_\alpha = \frac{1 - \alpha^2}{\alpha} \quad (\text{A.9.3-38})$$

$$\left(\frac{F_V}{Q_V}\right)_t = \frac{\alpha^2}{\alpha^2 + 1} \quad (\text{A.9.3-39})$$

Values of m_α and $(F_V/Q_V)_t$ for intermediate values of a can be solved for iteratively.

Selected values of $(F_V/Q_V)_t$ are provided in Table A.9.3-4:

Table A.9.3-4 — Values of $(F_V/Q_V)_t$ for various values of a and α

α	a						
	0,0	0,2	0,4	0,5	0,6	0,8	1,0
0,5	0,354	0,334	0,308	0,293	0,276	0,238	0,200
0,6	0,387	0,373	0,354	0,343	0,331	0,300	0,265
0,7	0,418	0,408	0,396	0,388	0,379	0,357	0,329
0,8	0,447	0,441	0,433	0,428	0,423	0,409	0,390
0,9	0,474	0,471	0,468	0,465	0,463	0,457	0,448
1,0	0,500	0,500	0,500	0,500	0,500	0,500	0,500

Selected values of m_α are provided in Table A.9.3-5:

Table A.9.3-5 — Values of m_α for various values of a and α

α	a						
	0,0	0,2	0,4	0,5	0,6	0,8	1,0
0,5	1,172	1,200	1,239	1,264	1,295	1,378	1,500
0,6	0,902	0,917	0,937	0,950	0,965	1,006	1,067
0,7	0,653	0,661	0,670	0,676	0,683	0,701	0,729
0,8	0,422	0,425	0,429	0,431	0,434	0,440	0,450
0,9	0,205	0,206	0,207	0,207	0,208	0,209	0,211
1,0	0,000	0,000	0,000	0,000	0,000	0,000	0,000

Equation (A.9.3-32) can be re-written to give the maximum moment on the spudcan as a function of the horizontal force as given in Equation (A.9.3-40):

$$F_M = f_2 Q_M \left[1 - \left(\frac{F_H}{f_1 Q_H} \right)^2 \right]^{0,5} \tag{A.9.3-40}$$

This equation applies only when the conditions given in Equations (A.9.3-41) and (A.9.3-42) are satisfied:

$$0 < \frac{F_V}{Q_V} < \left(\frac{F_V}{Q_V} \right)_t \tag{A.9.3-41}$$

and

$$F_H < f_1 Q_H \tag{A.9.3-42}$$

For a vertical and horizontal force combination that lies inside the yield surface given above, the moment on the spudcan is limited to the maximum available moment capacity Q_M .

A.9.3.3.4 Modification of the yield surface for partial penetration in sand

On seabeds of silica sands, conical spudcans that are not fully seated can develop increased moment capacity due to the rotation of the spudcan causing an eccentric seabed reaction which provides a beneficial resisting moment.

The effect may be taken into account for spudcans with $F_V/Q_V > 0,5$. The increased ultimate moment capacity Q_{Mp} due to eccentric seabed reaction is estimated as the minimum of Q_{Mps} and Q_{Mpv} , calculated from Equations (A.9.3-43) and (A.9.3-44) respectively; see Svaneø^[A.9.3-51]:

$$Q_{Mps} = 0,075 B Q_{Vnet} (B_{max}/B)^3 \quad (\text{A.9.3-43})$$

$$Q_{Mpv} = 0,15 B F_V \quad (\text{A.9.3-44})$$

Note that the horizontal capacity is unaffected.

The combined capacity should be checked against the modified yield interaction surface given in Equation (A.9.3-45):

$$\left(\frac{F_H}{Q_H}\right)^2 + \left(\frac{F_M}{Q_{Mp}}\right)^2 - 16 \left(\frac{F_V}{Q_V}\right)^2 \left(1 - \frac{F_V}{Q_V}\right)^2 = 0 \quad (\text{A.9.3-45})$$

A.9.3.3.5 Expansion of the yield surface for additional penetration in sand

Additional penetration of a spudcan in sands can be accounted for by using plasticity principles. Recommendations on updating stiffness and the flow of plastic displacements within a work-hardening framework are provided in Houlsby and Cassidy^[A.9.3-52], Cassidy *et al.*^[A.9.3-53] and Bienen *et al.*^[A.9.3-49].

This increase in penetration can also result in increased structural utilizations which should be assessed; see A.9.3.6.6.

A.9.3.3.6 Expansion of the yield surface for additional penetration in clay

For additional penetration of spudcans in clay, Wong and Murff^[A.9.3-36] and Van Langen and Hospers^[A.9.3-38] 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 rough flat-based circular rigid disk on an elastic half-space with modification factors to account for spudcan embedment. For the effects of leg embedment, see A.9.3.4.6. The elastic stiffness factors are calculated assuming full contact of the spudcan with the seabed. If the vertical reaction is insufficient to maintain full contact as the moment increases, then reduced stiffnesses should be used. The stiffness factors are derived for a homogeneous, linear, isotropic soil as given in Equations (A.9.3-46) to (A.9.3-48):

$$K_1 = K_{d1} \frac{2GB}{(1-\nu)} \quad (\text{vertical stiffness}) \quad (\text{A.9.3-46})$$

$$K_2 = K_{d2} \frac{16GB(1-\nu)}{(7-8\nu)} \quad (\text{horizontal stiffness}) \quad (\text{A.9.3-47})$$

$$K_3 = K_{d3} \frac{GB^3}{3(1-\nu)} \quad (\text{rotational stiffness for relatively low levels of loading; see Reference [A.9.3-54]})(\text{A.9.3-48})$$

Torsional spudcan foundation stiffness (i.e. for spudcan rotation about its vertical axis) should not be used.

The selection of the shear modulus of the foundation soil, G , is discussed in A.9.3.4.3 to A.9.3.4.5. An upper or lower bound value should be selected as appropriate for the analysis being undertaken, e.g. the upper value is appropriate for fatigue related analysis. The shear modulus is influenced by the stress level and strain amplitude. In general, the shear modulus decreases with increasing strain amplitude. In this part of ISO 19905, the consequences are addressed by reducing the stiffnesses.

NOTE Although the cross-coupling stiffness, K_4 , which links horizontal footing displacements and footing rotations to moment and horizontal loads, respectively, is not explicitly calculated, it is incorporated to some extent by the choice of the seabed reaction point as described in A.8.6.2.

A.9.3.4.2 Stiffness modifications

A.9.3.4.2.1 Embedment

Table A.9.3-6 provides values for the stiffness depth factors K_{d1} , K_{d2} and K_{d3} , to account for embedment effects on the stiffness of flat plate and conical type footings on an elastic half space, after Bell^[A.9.3-55]. Values for the case of partial backfill can be interpolated from the values for full and no backfill provided in the tables.

Table A.9.3-6 — Stiffness depth factors

Stiffness factors for $\nu = 0,0$						
2D/B	K_{d1}		K_{d2}		K_{d3}	
	No backfill	Full backfill	No backfill	Full backfill	No backfill	Full backfill
0,0	1,00	1,00	1,00	1,00	1,00	1,00
0,5	1,15	1,21	1,33	1,49	1,28	1,64
1,0	1,28	1,41	1,44	1,71	1,43	2,05
2,0	1,42	1,70	1,51	1,92	1,51	2,31
4,0	1,59	2,00	1,61	2,06	1,57	2,41

Stiffness factors for $\nu = 0,2$						
2D/B	K_{d1}		K_{d2}		K_{d3}	
	No backfill	Full backfill	No backfill	Full backfill	No backfill	Full backfill
0,0	1,00	1,00	1,00	1,00	1,00	1,00
0,5	1,11	1,18	1,32	1,47	1,23	1,54
1,0	1,21	1,34	1,42	1,67	1,37	1,90
2,0	1,34	1,59	1,48	1,85	1,44	2,15
4,0	1,49	1,85	1,58	1,98	1,51	2,25

Table A.9.3-6 (continued)

Stiffness factors for $\nu = 0,4$						
2D/B	K_{d1}		K_{d2}		K_{d3}	
	No backfill	Full backfill	No backfill	Full backfill	No backfill	Full backfill
0,0	1,00	1,00	1,00	1,00	1,00	1,00
0,5	1,08	1,14	1,31	1,45	1,18	1,43
1,0	1,16	1,27	1,41	1,64	1,31	1,76
2,0	1,27	1,48	1,48	1,80	1,39	2,01
4,0	1,41	1,72	1,57	1,92	1,47	2,13

Stiffness factors for $\nu = 0,5$						
2D/B	K_{d1}		K_{d2}		K_{d3}	
	No backfill	Full backfill	No backfill	Full backfill	No backfill	Full backfill
0,0	1,00	1,00	1,00	1,00	1,00	1,00
0,5	1,07	1,10	1,32	1,44	1,18	1,39
1,0	1,15	1,23	1,44	1,62	1,31	1,71
2,0	1,25	1,44	1,51	1,78	1,40	1,99
4,0	1,40	1,69	1,59	1,91	1,51	2,16

A.9.3.4.2.2 Cyclic loading

According to Andersen^[A.9.3-56], for clays (with OCR, $R_{OC} \geq 4$) subjected to cyclic actions, the cyclic foundation stiffnesses can be obtained by multiplying the static foundation stiffnesses with factors of 1,25 for horizontal, 1,25 for rotational and 3 to 8 for vertical stiffness. The reference static foundation 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.10.4.4.1.2 Option 1, the following procedure should be used if the reduction of rotational stiffness is not included in the soil model. The method accommodates stiffness reduction in a simple manner for responses within the yield surface.

If the force combination (F_V , F_H , F_M) lies outside the yield surface, the linearized rotational stiffness at the spudcan should be reduced using iterative analysis until the force combination lies on the yield surface.

Although the force combination (F_V , F_H , F_M) lies inside the yield surface, the initial estimate of linearized rotational stiffness should also be reduced by following the iterative procedure in A.10.4.4.1.2 and using the foundation rotational stiffness reduction factor, f_r , which has an increasing effect as the yield surface is approached. The factor is obtained from Equation (A.9.3-49); see Templeton^[A.9.3-57]:

$$f_r = (1 - n) r_f / \ln[(1 - nr_f)/(1 - r_f)] \quad (\text{A.9.3-49})$$

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 site. Finite element analysis for the Gulf of Mexico^[A.9.3-57] 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 rotational stiffness reduction factor expression takes the simpler form given in Equation (A.9.3-50):

$$f_r = -r_f / \ln(1 - r_f) \quad (\text{A.9.3-50})$$

As n approaches 1,0 the stiffness reduction expression tends towards the form given in Equation (A.9.3-51), which gives the most conservative treatment of stiffness reduction:

$$f_r = 1 - r_f \quad (\text{A.9.3-51})$$

The variable, r_f , in the stiffness reduction expression is the failure ratio defined by Equation (A.9.3-52):

$$r_f = \frac{\left[\left(\frac{F_H}{Q_H} \right)^2 + \left(\frac{F_M}{Q_M} \right)^2 \right]^{0,5}}{\left[16(1-a) \left(\frac{F_V}{Q_V} \right)^2 \left(1 - \frac{F_V}{Q_V} \right)^2 + 4a \left(\frac{F_V}{Q_V} \right) \left(1 - \frac{F_V}{Q_V} \right) \right]^{0,5}} \leq 1,0 \quad (\text{A.9.3-52})$$

where “ a ” is as defined in A.9.3.3.2.

NOTE $r_f > 1,0$ implies that the force combination (F_V, F_H, F_M) lies outside the yield surface. Under such conditions, the reduced stiffness factor is not applicable, and the rotational stiffness is reduced until the force combination lies on the yield surface.

For fully embedded foundations in clays at vertical force ratio $F_V / Q_V < \left(\frac{F_V}{Q_V} \right)_t$, the failure ratio can be expressed as given in Equation (A.9.3-53):

$$r_f = \left[\left(\frac{F_H}{f_1 Q_H} \right)^2 + \left(\frac{F_M}{f_2 Q_M} \right)^2 \right]^{0,5} \leq 1,0 \quad (\text{A.9.3-53})$$

where $\left(\frac{F_V}{Q_V} \right)_t$, f_1 and f_2 are as defined in A.9.3.3.3.

A.9.3.4.2.4 Non-linear vertical, horizontal and rotational stiffness

A full yield interaction surface model that includes non-linear vertical, horizontal and rotational stiffnesses implicitly incorporates the necessary stiffness reduction as a consequence of work-hardening plastic displacement and rotation (van Langen *et al.*^[A.9.3-58], Wong *et al.*^[A.9.3-59], and Cassidy *et al.*^[A.9.3-60]). The stiffness reduction factor should not be applied.

A.9.3.4.2.5 Non-linear continuum foundation model

A continuum foundation model that includes non-linear soil behaviour (e.g. elastic-plastic work hardening) implicitly incorporates the necessary stiffness reduction. The stiffness reduction factor should not be applied.

A non-linear continuum foundation model should not be used unless one of the simpler analysis methods has been used to provide a benchmark for the results.

A.9.3.4.3 Selection of shear modulus, G , for clay

The value of the initial, small-strain shear modulus for clay, G , should be based on the value of the undrained shear strength, s_u , measured at the depth $z = D + 0,15B$, where B is the effective diameter of the spudcan in contact with the soil and D is the predicted depth below the sea floor of the lowest point on the spudcan with diameter B . Where the clay is significantly layered, the average strength within the range $z = D$ to $z = D + 0,3B$ should be used. Except in areas with carbonate clays or clayey silts the shear modulus should be calculated from Equation (A.9.3-54), see References [A.9.3-61] and [A.9.3-62]:

$$G = G_{\max} = s_u \frac{600}{R_{OC}^{0,25}} \quad \text{with } G < s_u I_{rNC} \text{ and subject to the limitations given below.} \quad (\text{A.9.3-54})$$

where

G_{\max} is the maximum value of the shear modulus, which occurs at small strain;

NOTE 1 In forming estimates of foundation stiffness from linear elastic solutions to represent non-linear soil behaviour, one general method uses the linear elastic stiffness solution with a shear modulus taken as a function of strain level. Another method uses a non-linear stiffness function, which varies with the amplitude of the action and a constant shear modulus. In the former method a distinction is made between the term, G_{\max} (the maximum value of the shear modulus, which occurs at small strain) and the term G (the general shear modulus, which varies with strain magnitude). In the latter method, the maximum value of shear modulus is used and no such distinction in terms is made. Consequently, in this part of ISO 19905, the term G should be taken to refer to the maximum value, which occurs at small strain.

R_{OC} is the overconsolidation ratio;

I_{rNC} is the rigidity index for normally consolidated clays.

For extreme loading situations, and in the absence of other data, I_{rNC} should be conservatively limited to 400; see Noble Denton[A.9.3-62].

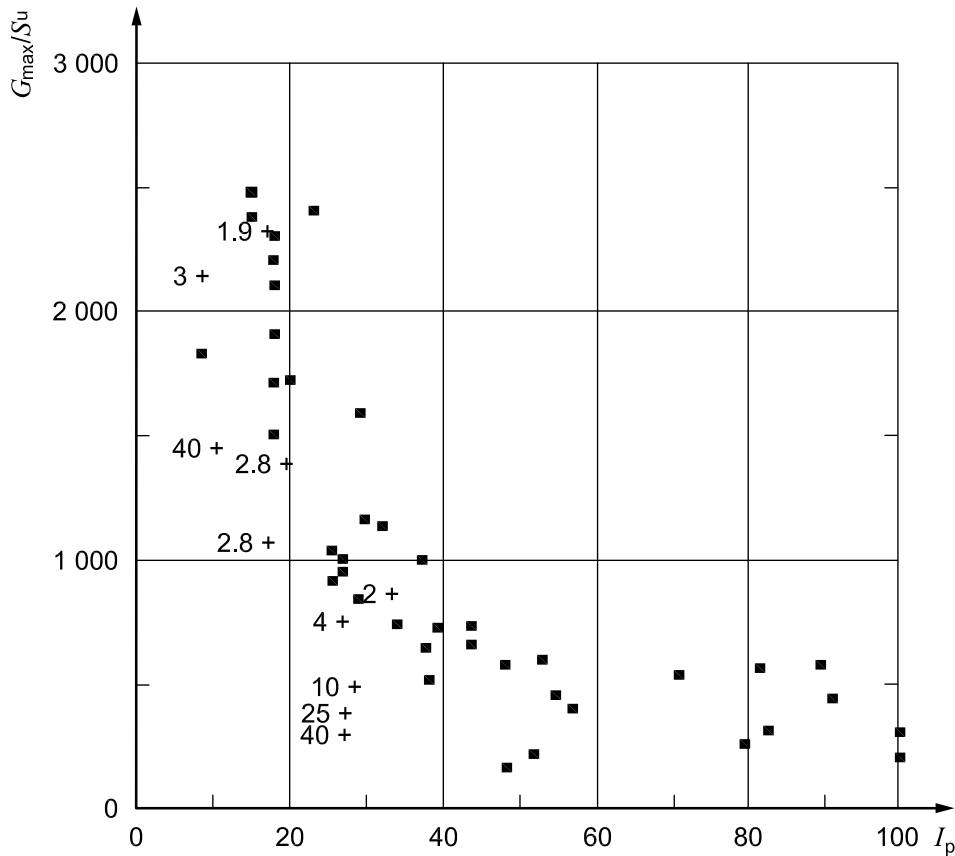
NOTE 2 The recommendations of Reference [A.9.3-62] are based on overconsolidated clays with plasticity indices of up to 60 %. Due consideration should be given to the possibility of determining site-specific shear moduli for cohesive soils other than overconsolidated clays and/or where the plasticity indices exceed 60 %.

$I_{rNC} = 600$ is supported by field data for jack-up response in the Gulf of Mexico; see Templeton[A.9.3-41].

In some cases, higher ratios of I_{rNC} have been reported. The data in Figure A.9.3-16 support the use of higher values (possibly between 1 000 and 2 500) for plasticity indices less than 20 %.

It should be recognized that I_{rNC} generally decreases with increasing plasticity index (Andersen[A.9.3-56], Figure 10.2; reproduced as Figure A.9.3-16). For clays with plasticity indices less than 20 % or greater than 60 % and where the shear modulus is not supported by site-specific data, the assessor should account for this trend when determining G .

The recommendations given above (Cassidy *et al.*[A.9.3-61]) 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 equations given in A.9.3.4.2.3, or implicitly using non-linear foundation models. In the case of small-strain applications such as in structural fatigue analysis, the stiffness reductions do not apply, and it can be appropriate to adopt upper-bound values of G .



Key

+ $R_{oc} > 1,5$ (with R_{oc} value indicated at data point)

■ $1,0 < R_{oc} < 1,5$

I_p plasticity index in percent

R_{oc} over-consolidation ratio

G_{max} maximum soil shear modulus

s_u undrained shear strength from direct simple shear test

NOTE 1 The determination of G_{max} via the use of rigidity index is inherently approximate. The variability in the shear strength determined by different test methods is not significant in comparison to the uncertainty in this approximation. The approach can, therefore, be used when direct simple shear test results are not provided.

NOTE 2 Taken from Andersen^[A.9.3-56], Figure 10.2.

Figure A.9.3-16 — Normalized initial shear modulus as a function of plasticity index, I_p , for 11 different clays

A.9.3.4.4 Selection of shear modulus, G , for sand

For sands, the initial small-strain shear modulus should be computed from Equation (A.9.3-55):

$$G/p_a = j(V_{sw}/Ap_a)^{0,5} \tag{A.9.3-55}$$

where

j is the dimensionless stiffness factor, $j = 230 \left(0,9 + \frac{D_R}{500} \right)$;

p_a is the atmospheric pressure, typically taken as 101,3 kPa;

D_R is the relative density (expressed in percent);

V_{sw} is the gross vertical spudcan reaction inclusive of backfill under still water conditions (the reaction that would be obtained if the jack-up were supported on an infinitely rigid foundation, plus the reaction due to the submerged weight of any backfill on the spudcan, less the submerged weight of soil displaced by the spudcan below D , the greatest depth of maximum cross-sectional spudcan bearing area below the sea floor).

The recommendations given above (Cassidy *et al.*^[A.9.3-61]) 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 equations given in A.9.3.4.2.3, or implicitly using non-linear foundation models. In the case of small-strain applications such as in structural fatigue analysis, the stiffness reductions do not apply, and it can be appropriate to adopt upper-bound values of G .

A.9.3.4.5 Selection of shear modulus for layered soils

Roesset^[A.9.3-63] 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^[A.9.3-64].

A.9.3.4.6 Soil-leg interaction

For deep penetrations, typically experienced in soft clay conditions, the calculation of foundation fixity can be augmented with the inclusion of the lateral soil resistance on the leg members (Brekke *et al.*^[A.9.3-65]).

The lateral soil resistance of the backfill material can be modelled based on concepts proposed by Matlock^[A.9.3-66] for lateral soil resistance of piles. The jack-up leg can be modelled as an equivalent pile for purposes of determining p - y , or load-deflection curves.

The diameters of the individual members (i.e. leg chords and braces) give appropriate characteristic dimensions for determining the p - y curves. The p - y curves for each member are directionally combined to form equivalent p - y curves along the leg, accounting for soil layering and changes in leg geometry. Any external face of each leg in compressive contact with the soil may be assumed to contribute to the lateral resistance. Typically, equivalent springs at each bay elevation are used to simplify the calculations.

A.9.3.5 Vertical-horizontal foundation capacity envelopes

A.9.3.5.1 General ultimate vertical-horizontal foundation capacity envelope

The general gross ultimate vertical-horizontal foundation capacity envelope for jack-up spudcans is a two-dimensional slice of the full vertical-horizontal-moment envelope as given in A.9.3.3.2. If the spudcan moment capacity is zero (i.e. $F_M = 0$), the ultimate vertical-horizontal foundation capacity envelope is as given in Equation (A.9.3-56):

$$\left(\frac{F_H}{Q_H}\right)^2 - 16(1-a)\left(\frac{F_V}{Q_V}\right)^2 \left(1 - \frac{F_V}{Q_V}\right)^2 - 4a\left(\frac{F_V}{Q_V}\right)\left(1 - \frac{F_V}{Q_V}\right) = 0 \quad (\text{A.9.3-56})$$

For small embedments (in the limit as $a \rightarrow 0$), this equation reduces to Equation (A.9.3-57):

$$\left(\frac{F_H}{Q_H}\right) - 4\left(\frac{F_V}{Q_V}\right)\left(1 - \frac{F_V}{Q_V}\right) = 0 \quad (\text{A.9.3-57})$$

where Q_V is taken to be equal to the gross ultimate vertical foundation capacity of the soil beneath the spudcan (achieved during preloading), evaluated as described in A.9.3.2.2 to A.9.3.2.6, and Q_H as defined in A.9.3.3.2.

A.9.3.5.2 Ultimate vertical-horizontal foundation capacity envelopes for spudcans in sand

The yield surface used for checking the vertical-horizontal foundation capacity of spudcans in sand is presented in A.9.3.5.1.

The sliding failure envelope used for checking the sliding capacity of a spudcan in sand is as given in Equation (A.9.3-58):

$$Q_{Hs} = F_V \tan \delta + 0,5 \gamma' (k_p - k_a) (h_1 + h_2) A_s \quad (\text{A.9.3-58})$$

where

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

$$F_V = V_{st} - B_S \quad (\text{with no backfill})$$

$$F_V = V_{st} + W_{BF,o} + W_{BF,A} - B_S \quad (\text{with backfill}) \quad (\text{A.9.3-59})$$

h_1 is the embedment depth to the uppermost part of the spudcan, (if not fully embedded $h_1 = 0$);

h_2 is the spudcan tip embedment depth;

k_a is the active earth pressure coefficient (for $s_u = 0$), $k_a = \tan^2(45 - \phi/2)$;

k_p is the passive earth pressure coefficient, $k_p = 1/k_a$;

δ is the steel/soil friction angle in degrees:

$$\delta = \phi' - 5^\circ \quad (\text{for a flat-bottom spudcan, } \beta = 180^\circ),$$

$$\delta = \phi' - 0,5 (\beta - 170^\circ) \quad (\text{for } 170^\circ < \beta < 180^\circ), \quad (\text{A.9.3-60})$$

$$\delta = \phi' \quad (\text{for a conically shaped spudcan, } \beta \leq 170^\circ)$$

where

β is the effective cone angle in degrees (see Figure A.9.3-3);

ϕ' is the effective angle of internal friction for sand in degrees.

A.9.3.5.3 Ultimate vertical-horizontal foundation capacity envelopes for spudcans in clay

The yield surface used for checking the vertical-horizontal foundation capacity for spudcans in clay for $F_V > 0,5 Q_V$ is presented in A.9.3.5.1 and for $F_V < 0,5 Q_V$ in A.9.3.3.3.

The sliding capacity, Q_{Hs} , in clay can be assumed to be Q_H as defined in A.9.3.3.2.

A.9.3.5.4 Ultimate vertical-horizontal foundation capacity envelopes for spudcans on layered soils

The foundation capacity of layered soils can be determined using the principles of limiting equilibrium analysis or the finite element method. Alternatively, the equations given in A.9.3.5.2 and A.9.3.5.3 can be used to make a conservative estimate of the ultimate vertical-horizontal capacity relationship for layered soils by considering failure through the weakest zones in such a soil profile.

A.9.3.6 Acceptance checks

A.9.3.6.1 General

Figure A.9.3-17 shows the overall approach to the foundation acceptance checks.

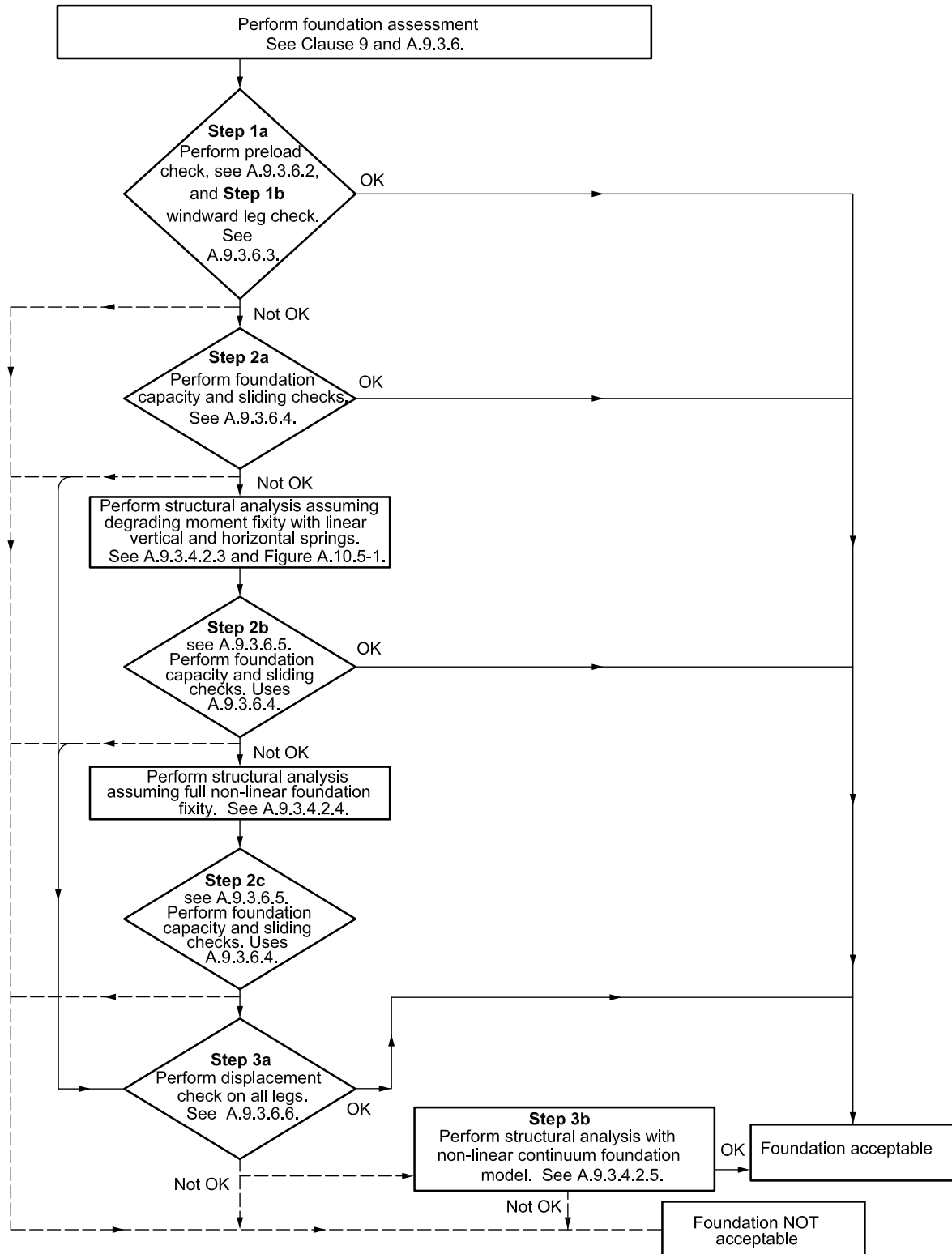


Figure A.9.3-17 — Approach to foundation acceptance checks

A.9.3.6.2 Level 1, Step 1a — Ultimate bearing capacity check for vertical loading of the leeward leg - preload check (pinned spudcan)

The preload check should be applied only when the horizontal force on the leeward leg spudcan, F_H , is no greater than F_{H1} (see Table A.9.3-7) and when the forces are determined from an analysis model with pinned condition for all spudcans. In this case, the maximum gross vertical force F_V should comply with the limit given in the applicable Equation (A.9.3-61) or Equation (A.9.3-62):

$$F_V \leq V_{Lo} \gamma_{R,PRE} - B_S \quad (\text{with no backfill}) \quad (\text{A.9.3-61})$$

$$F_V \leq V_{Lo} \gamma_{R,PRE} \pm W_{BF,o} - B_S \quad (\text{with backfill}) \quad (\text{A.9.3-62})$$

where

$\gamma_{R,PRE}$ is the preload resistance factor, $\gamma_{R,PRE} = 1,10$;

$W_{BF,o}$ is the submerged weight of any backflow and infill that is predicted to occur during preloading;

F_V is the gross vertical force acting on the soil beneath the spudcan due to the assessment load case F_d (see 8.8) as given in Equations (A.9.3-63):

$$F_V = V_{st} - B_S \quad (\text{with no backfill})$$

$$F_V = V_{st} + W_{BF,o} + W_{BF,A} - B_S \quad (\text{with backfill}) \quad (\text{A.9.3-63})$$

V_{st} is the vertical force applied to the spudcan due to the assessment load case F_d (see 8.8). This includes quasi-static contributions due to factored actions, and contributions from dynamic response, as appropriate, in accordance with the procedures of Clause 10, and also includes leg weight and water buoyancy but excludes the submerged weight of backfill ($W_{BF,o} + W_{BF,A}$) and the soil buoyancy of the spudcan below the bearing area B_S ;

$W_{BF,A}$ is the submerged weight of any backflow and infill that is predicted to occur after the maximum preload has been applied and held.

Table A.9.3-7 — Limiting horizontal capacity, F_{H1} , for Step 1a bearing capacity check

Soil type	Embedment	Limiting horizontal capacity, F_{H1} for Step 1a to apply
Sand	Partial	$[0,1 - 0,07 (B/B_{max})^2] Q_{Vnet}$
	Full	$0,03 Q_{Vnet}$
Clay	Any	$0,03 Q_{Vnet}$

NOTE 1 The constants in Equations (A.9.3-61) and (A.9.3-62) include the effects of $\gamma_{R,PRE} = 1,10$. The limiting horizontal capacity, F_{H1} , for Step 1a was determined from the intersection between unfactored vertical-horizontal bearing capacity envelope and maximum allowable gross vertical reaction, $Q_{V,max}$, with some reduction applied for conservatism. Assuming $a = 0$, the limiting horizontal capacity, F_{H1} , corresponding to maximum vertical capacity, $Q_{V,max}$, can be computed from Equation (A.9.3-64):

$$F_{H1} = 4 \left[1 - \frac{(\gamma_{R,PRE} - 1)V_{Lo}}{\gamma_{R,PRE} Q_V} \right] \left[\frac{(\gamma_{R,PRE} - 1)V_{Lo}}{\gamma_{R,PRE} Q_V} \right] Q_H \quad (\text{A.9.3-64})$$

For $\gamma_{R,PRE} = 1,10$, Equation (A.9.3-64) can be approximated by $F_{H1} \approx 0,33 V_{Lo} Q_H/Q_V$ and is equivalent to $F_{H1} \approx 0,04 Q_{Vnet}$ for shallow penetrations in sand. Conservatively, 0,03 was used for the limits given in Table A.9.3-7 (see also Note 2).

NOTE 2 For shallow spudcan penetrations and vertical reaction of $0,9 Q_{Vnet}$, the available unfactored horizontal capacity is approximately $0,04 Q_{Vnet}$. If the horizontal reaction exceeds $0,04 Q_{Vnet}$, additional penetration can occur. The use of $0,03 Q_{Vnet}$ in the check, therefore, includes a level of conservatism. If the spudcan is fully embedded, the additional penetration can be significant. Additional penetration can increase the soil resistance but, to increase the horizontal capacity to $0,1 V_{Lo}$, the additional penetration is about 10 % of the spudcan diameter and outside tolerable limits. Conversely, where the spudcan is partially embedded (i.e. when the maximum spudcan bearing area is not mobilized), any additional penetration results in a significant increase of bearing capacity due to the rapid increase in the bearing area. An increase in embedded area of approximately 10 % increases the vertical bearing capacity such that, simultaneously, the horizontal foundation capacity increases to $0,1 V_{Lo}$.

NOTE 3 For partial spudcan penetration in sand, Q_{Vnet} can be taken as being equal to V_{Lo} for the purposes of the Step 1a check.

A.9.3.6.3 Level 1, Step 1b — Check of the windward leg — Pinned spudcan

The windward leg check should be applied only when the horizontal force on the windward leg spudcan, F_H , is no greater than F_{H1} (see Table A.9.3-7). In this case, the sliding stability of the windward leg is checked by ensuring that the vertical reaction complies with Equation (A.9.3-65):

$$F_V > (1 - 1/\gamma_{R,PRE}) Q_V \quad (\text{A.9.3-65})$$

where $\gamma_{R,PRE}$ is the preload resistance factor, $\gamma_{R,PRE} = 1,10$.

In the case of a sand foundation, this check is valid for sand friction angle $\phi' \geq 25^\circ$. For friction angles $\phi' < 25^\circ$, the sliding check in Step 2 should be performed.

A.9.3.6.4 Level 2, Step 2a — Foundation capacity and sliding check — Pinned spudcan

A.9.3.6.4.1 Step 2a — Foundation capacity check

A reduction in the ultimate vertical bearing capacity, Q_V , of a spudcan foundation occurs when it is simultaneously subjected to a horizontal force, F_H , and a moment, F_M . The latter is ignored in Step 2a analyses as the spudcans are considered to be pinned. The following paragraph describes the construction of the factored vertical-horizontal foundation capacity envelope and the foundation capacity check for Step 2a which is also applicable to Step 2b.

The vertical-horizontal foundation capacity for sands and clays can be generated according to A.9.3.5 and the spudcan reactions should be evaluated for each spudcan. If the reaction forces on the spudcan lie within the factored vertical-horizontal bearing capacity envelope and the factored sliding failure envelope (see A.9.3.6.4.2), the foundation is satisfactory. To obtain the factored vertical-horizontal bearing envelope, the vertical-horizontal capacity envelope is scaled by the resistance factor, $\gamma_{R,VH}$, from the point of zero net reaction, i.e., $(F_H = 0, F_V = W_{BF,0} - B_S)$. In effect, the envelope is shrunk towards this scaling origin.

A measure of the foundation utilization (see Clause 13) can be obtained by assessing the proximity of the loading point (F_H, F_V) to the factored vertical-horizontal bearing capacity envelope. When making the check, the magnitude of the vector to the loading point should be compared against the magnitude of the vector to the factored vertical-horizontal bearing capacity envelope. The origin of the vectors is arbitrary; however, for consistency and to help produce a meaningful value of the resulting utilization, the origin of the vectors $(F_H, F_V)_{ORG}$ should be taken on the vertical capacity axis (at zero shear) at $0,5 Q_V/\gamma_{R,VH}$ (see Figure A.9.3-18). Accordingly, each spudcan foundation should satisfy the capacity check given in Equation (A.9.3-66):

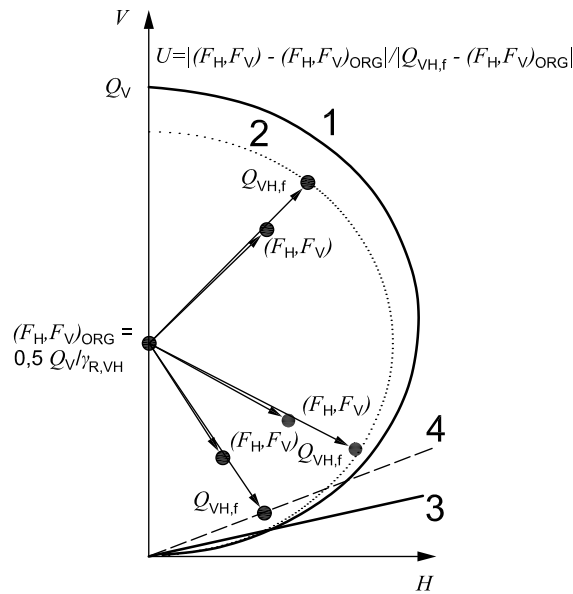
$$|(F_H, F_V) - (F_H, F_V)_{ORG}| \leq |Q_{VH,f} - (F_H, F_V)_{ORG}| \quad (\text{A.9.3-66})$$

where

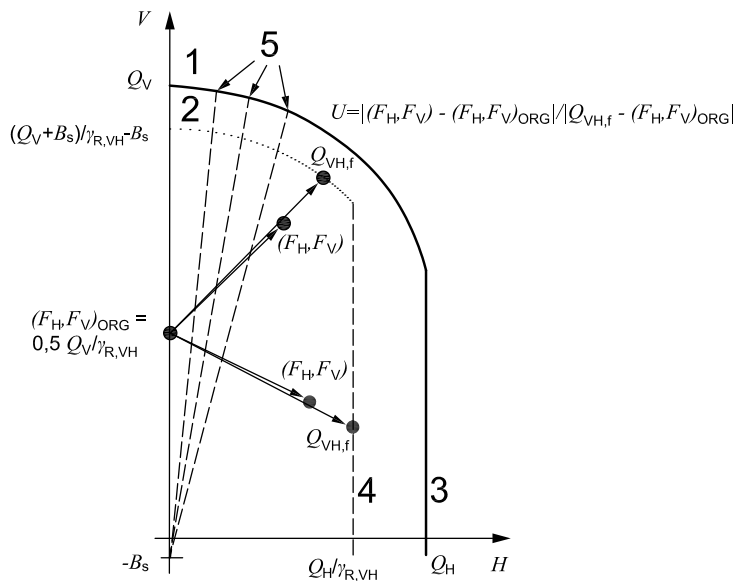
(F_H, F_V) is the environmental response point (determined from factored actions);

$(F_H, F_V)_{ORG}$ is the origin used for establishing the utilization; this should be taken as $H = 0,0$;
 $V = 0,5 Q_V/\gamma_{R,VH}$;

- Q_V is the gross ultimate vertical foundation capacity;
- $Q_{VH,f}$ is the point where the vector originating from $(F_H, F_V)_{ORG}$ and passing through (F_H, F_V) intersects the applicable factored vertical-horizontal capacity surface. The factored vertical-horizontal capacity surface is derived by dividing the coordinates of the applicable surface from A.9.3.5 by the resistance factor $\gamma_{R,VH}$ with respect to the point of zero net reaction $(0, W_{BF,0} - B_S)$;
- $\gamma_{R,VH}$ is the partial resistance factor for vertical-horizontal foundation bearing capacity, $\gamma_{R,VH} = 1,10$;
- $|\dots|$ represents the vector magnitude.

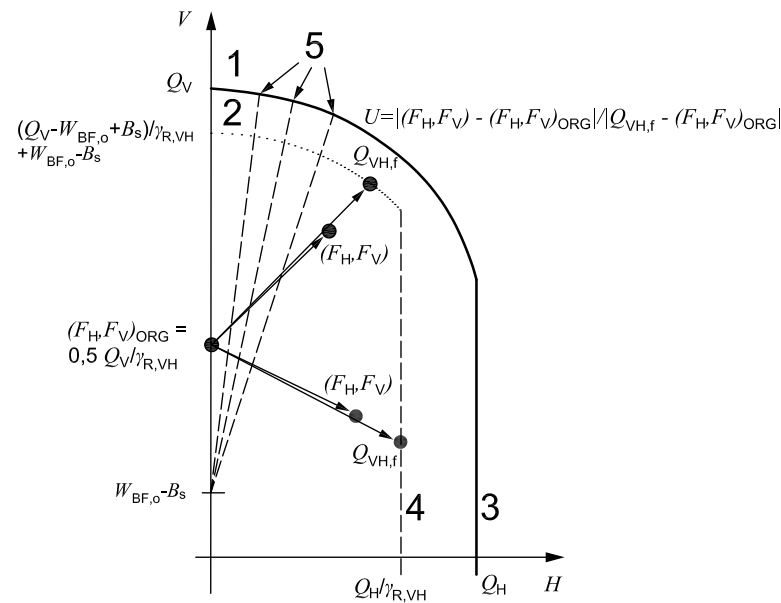


a) Sand



b) Clay with spudcan buoyancy and no backfill

Figure A.9.3-18 (continued)



c) Clay with spudcan buoyancy and backfill

Key

- 1 vertical-horizontal foundation capacity
- 2 factored vertical-horizontal foundation capacity (coordinates multiplied by $1/\gamma_{R,VH}$) relative to the scaling origin as defined
- 3 sliding capacity (see A.9.3.6.4.2)
- 4 factored sliding capacity (unfactored horizontal sliding capacity coordinate multiplied by $1/\gamma_{R,Hfc}$)
- 5 vectors indicating origin for construction of the factored V-H bearing capacity envelope
- $|\dots|$ represents the vector magnitude
- H horizontal reaction or horizontal capacity
- Q_V gross ultimate vertical foundation capacity (with zero horizontal load)
- $Q_{VH,f}$ point where the vector originating from $(F_H, F_V)_{ORG}$ and passing through (F_H, F_V) intersects the factored vertical-horizontal capacity surface derived by dividing the coordinates of the applicable surface from A.9.3.5 by the resistance factor $\gamma_{R,VH}$
- U utilization for environmental response point (F_H, F_V) as given in A.9.3.6.4
- V vertical reaction or vertical capacity
- $\gamma_{R,VH}$ partial resistance factor for foundation (bearing) capacity
- $\gamma_{R,Hfc}$ partial resistance factor for horizontal (sliding) capacity

Figure A.9.3-18 — Vertical-horizontal foundation capacity envelopes

A.9.3.6.4.2 Step 2a — Foundation sliding check

In Step 2a, the spudcan foundations should also be assessed using the following sliding check, since the factored sliding failure surface can lie within the factored vertical-horizontal bearing capacity envelope. The same procedure also applies for Step 2b.

The horizontal capacity of the foundations of the windward leg(s) should be checked for the horizontal forces on the spudcan(s), F_H , in association with the gross vertical force F_V . 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 capacity check given in Equation (A.9.3-67):

$$|(F_H, F_V) - (F_H, F_V)_{\text{ORG}}| \leq |Q_{\text{VH},f} - (F_H, F_V)_{\text{ORG}}| \quad (\text{A.9.3-67})$$

where

$Q_{\text{VH},f}$ is the point where the vector originating from $(F_H, F_V)_{\text{ORG}}$ and passing through (F_H, F_V) intersects the applicable factored sliding capacity surface derived by dividing the horizontal coordinates of the applicable surface Q_{Hs} from A.9.3.5.2 (sand) or A.9.3.5.3 (clay) by the resistance factor, $\gamma_{\text{R,Hfc}}$;

Q_{Hs} is the foundation sliding capacity; see A.9.3.5.2 (sand) or A.9.3.5.3 (clay);

$\gamma_{\text{R,Hfc}}$ is the partial resistance factor for horizontal foundation capacity where $\gamma_{\text{R,Hfc}} = 1,25$ for sand, based on drained conditions and effective stress, or $\gamma_{\text{R,Hfc}} = 1,56$ for clay, based on undrained conditions and total stress;

$|\dots|$ represents the vector magnitude.

A.9.3.6.5 Level 2, Steps 2b and 2c — Foundation capacity and sliding check — Spudcan with moment fixity and vertical and horizontal stiffness

Step 2b and 2c foundation analyses inherently ensure 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 undertake in a Step 2b assessment are identical to those undertaken for Step 2a in which vertical-horizontal capacity and sliding capacity for sands and clays can be generated in accordance with A.9.3.5 and the spudcan reactions are evaluated for each spudcan. If the vertical and horizontal reactions from the response analysis (which has accounted for spudcan moment fixity with stiffness reduction) lie within the factored foundation capacity envelopes, the foundation is satisfactory.

A Step 2c analysis implicitly includes a check on compliance with the unfactored foundation yield surface. When the frictional sliding surface intersects the foundation capacity envelope, sliding can occur before the response reaches the yield surface. When this sliding effect is included in the response analysis, no further Level 2 checks are required. When this sliding effect is not included, a sliding check should be undertaken in accordance with A.9.3.6.4.2. In all cases, the Level 3, Step 3a displacement check should be performed.

A.9.3.6.6 Level 3, Steps 3a and 3b — Displacement check — Settlements resulting from exceedance of the foundation capacity

Vertical settlement and/or sliding of a spudcan can occur if the forces on the spudcan due to the extreme event are outside the yield interaction surface computed for the spudcan at the penetration achieved during installation. Such settlements often result in a gain in capacity through expansion of the yield interaction surface. However, the integrity of the foundation can decrease in the situation where a potential punch-through exists, e.g., where dense sand overlies soft clay. More thorough analyses should be performed for such cases and for the complex and/or potentially dangerous foundation conditions listed in A.9.3.2.5 and A.9.3.2.6.

A Step 3a check can be accomplished by identifying the “equivalent” preload level that would be required to expand the V - H yield surface used in Step 2 such that the factored capacity exceeds the forces on the spudcan. The added penetration associated with this “equivalent” preload is calculated using each of the three predicted load-penetration curves [using the best estimate, upper bound and lower bound soil strength profiles and separate global response analyses as appropriate; see A.9.3.2.1.1 b)]. If any of these three additional penetrations is significant, the effects on the spudcan foundation and the structure should be evaluated and the procedure iterated to establish whether the consequences of the displacement on the other utilization checks are acceptable.

A Step 3a check can also be performed when the Level 2a or 2b sliding or capacity check of a windward leg is not satisfied, or is no longer satisfied due to the additional penetration of a leeward leg as described above. In the case of a windward leg, sliding can occur when the factored load exceeds the factored capacity resulting in redistribution of the horizontal reaction to the leeward leg foundations. This effect can be assessed by limiting the factored horizontal reaction to the factored sliding limit (dependent on F_V) and iteratively determining the redistribution of loads and the associated non-linear displacement of the structure. The effects on all spudcan foundations and the structure should be evaluated and the procedure iterated to establish whether the consequences of the displacement on all the other utilization checks are acceptable, including the foundation capacity of the other legs.

A Step 3b analysis inherently includes a check on the direct consequences of spudcan displacement. Therefore, no foundation checks are required, although it should be shown that the results are not sensitive to the load-penetration assumptions, i.e. that small changes in the forces on the spudcan or assumed soil strength do not lead to large increases in penetration.

When assessing the acceptability of displacements, due consideration should be given to operational limitations, e.g. jacking operations to level the unit and re-establish a safe hull elevation or to depart the site. The limits are dependent upon the jack-up and the configuration at the site.

A.9.3.6.7 Foundation settlement not specifically addressed elsewhere

Settlement of the spudcans should be estimated and checked. If necessary, corrective actions should be taken. The settlements of installed spudcans can be assessed from a combination of

- elastic settlements;
- consolidation settlement;
- settlements due to cyclic loading;
- settlements due to seabed instability.

The elastic settlements and consolidation settlements can be calculated using conventional analytical or numerical geotechnical models. The elastic settlements occur concurrently with applied actions and can be calculated as function of the basic elastic soil properties (ν and G) and the applied actions. The consolidation settlements of cohesive soils can be calculated using conventional models accounting for time effects.

Cyclic environmental actions or operational vibrations can induce further settlements. Special attention should be given to cyclic loading in silty sand or silt. Cyclic loading can also involve a soil strength reduction. This can induce settlements due to bearing failure.

Seabed instability due to scour or gas seeps involves a decrease in the effective bearing capacity. This can induce settlements due to local bearing failure.

The settlements should be checked regularly. If necessary level adjustments should be made or protective measures against scour development should be taken (see A.9.4.7).

A.9.4 Other considerations

A.9.4.1 Skirted spudcans

Skirts are added to spudcans to provide additional foundation capacity and stiffness compared to conventional conical spudcan geometries.

Within the skirt, the typical geometry of the underside of a skirted spudcan is either relatively flat or conical. In some cases, the leg chords may protrude below the skirt tip and achieve first contact with the sea floor, thus protecting the skirt whilst going on and off location. In the case of skirted spudcans with a flat underside, a

level and undisturbed seafloor surface is required in order to minimize the potential for eccentricity of the foundation reaction.

In order to realise the maximum benefit from using a skirted spudcan, the underside of the skirted spudcan should achieve full contact with the sea floor surface. Calculations should be performed to determine the penetration resistance of the skirt, including any bulkheads and internal or external stiffeners and, hence, whether the applied preload is sufficient to ensure that full contact is achieved.

Methods for calculating the tip and skin friction components of the skirt penetration resistance are described in DNV Classification Note 30.4^[A.9.4-1]. In cases where the skirt tip has a greater thickness than the rest of the skirt, consideration should be given to the potential for a gap to form above the skirt tip during penetration into the seabed, especially in cohesive soils.

If the penetration resistance exceeds the available preload footing reaction and partial penetration of the skirt occurs, consideration can be given to measures such as applying suction for increasing the penetration or infilling the resulting void within the skirt with suitable material introduced through valved pipes that penetrate into the skirt void. If, after preloading, the skirt is partially penetrated, the assessment should be revised to determine the consequences, including a consideration of the strength of the skirt.

Consideration should also be given to the effects of compaction and/or consolidation of the soils within the skirt or any infill material used during preloading.

If the voids within the skirt are not completely infilled, consideration should be given to the effect of movement of the enclosed seawater within the skirt due to spudcan rotation, especially for compartmentalized skirts in cohesionless soils where “piping” can occur due to flow of the enclosed water around stiffener plates or bulkheads.

Once full contact has been achieved, the vertical bearing capacity of the skirted spudcan essentially corresponds to that of an embedded footing. As the soil within the spudcan skirt is effectively part of the spudcan, the weight of the enclosed soil plug should be incorporated in the penetration resistance calculations.

At locations with relatively strong soils and when the underside of the skirt spudcan is flat, the ultimate bearing capacity of the foundation can be significantly greater than the applied preload. Methods are available for the determination of such additional “virtual” capacities, see for example DNV Classification Note 30.4^[A.9.4-1].

The bearing capacity envelopes appropriate for skirted footings have been the subject of much research; see Dean *et al.*^[A.9.3-34], Bransby and Randolph^{[A.9.4-2] [A.9.4-3]}, Bransby and Yun^[A.9.4-4], Cassidy *et al.*^[A.9.4-5], Eide *et al.*^[A.9.4-6], Gourvenec^{[A.9.4-7] [A.9.4-8]}, Gourvenec and Randolph^[A.9.4-9], Kellezi *et al.*^{[A.9.4-10] [A.9.4-11] [A.9.4-12]}, Leland *et al.*^[A.9.4-13] and Svano and Tjelta^[A.9.4-14]. The skirted spudcan has generally been modelled as a solid footing, however, care is warranted before making such an assumption as weaker soil from the seabed surface trapped within the spudcan skirt can influence the failure mechanisms developed, reducing the additional capacity available; see Bransby and Yun^[A.9.4-4].

When full spudcan-seabed contact is achieved, the embedment of the skirted spudcan can permit the use of elastic foundation stiffness depth factors corresponding to a solid footing as described in Bell^[A.9.3-55].

The extraction resistance for a skirted spudcan can be substantial; consequently, skirted spudcans are not usually employed at locations where soil backfill can occur on top of the spudcans. Extraction can be assisted through the use of drainage and/or the application of water pressure within the skirt in order to minimize the development of suction within the soil below the spudcan.

Soil can remain within the skirts after extraction of a skirted spudcan from a location with cohesive soils, which can influence the penetration response during subsequent installations.

A.9.4.2 Hard sloping strata

A hard, sloping stratum can be created by a sand wave, sand bank, scour around a platform, buried geomorphic features such as channels, footprints produced by previous jack-up emplacements, human-related seabed activity, or a combination of the above. Such slopes can cause eccentricity in the spudcan

reaction, which can lead to emplacement and removal difficulties, particularly for leg designs with slender braces, as in the following examples:

- The eccentric reaction can result in a significant leg bending moment in the region of the hull. Where this bending moment is reacted by the leg guides, the resulting large shear force can overstress the leg members.
- If a fixation system (rack chocks) is employed at the leg-to-hull interface, the bending moment present at the time when the fixation system is engaged is locked into the leg. If the eccentricity of the spudcan reaction is subsequently exacerbated (e.g. by scouring around the spudcan), then the effective leg bending moment in the region of the hull can increase. When the fixation system is later disengaged, the redistribution of the moment in the leg for the revised support condition provided by the pinions and guides can cause overstress.

Anticipated installation-induced stresses should be considered in the site-specific assessment (see 5.4.8). The foundation reactions should be assessed against bearing and sliding capacities of the sloping hard stratum.

Consideration can be given to the potential benefit of seabed preparation prior to emplacement of the jack-up.

A.9.4.3 Footprint considerations

Surface or buried footprints from prior jack-up operations in the proposed field can cause eccentric reactions or lateral movement of the spudcan. One preventive approach is avoidance (i.e. positioning spudcans at some minimum distance away from the footprints) while mitigations include working the legs, leg stomping, seabed remediation, etc.

Information on spudcan-footprint interaction can be found in References [A.9.4-15] to [A.9.4-22].

A.9.4.4 Leaning instability

A lower bound estimate of the leaning stability can be obtained using the theory of Hambly^[A.9.4-23]. 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.9.4.5 Leg extraction difficulties

Leg extraction difficulties can be caused by conditions including the following:

- deeply penetrated spudcan in soft clay or loose silt;
- skirted or caisson-type spudcan where uplift resistance can be greater than the installation reaction;
- sites where the soil exhibits increased strength with time.

A jack-up pulls its legs from the seabed by lowering the hull into the water, thereby generating a buoyant uplift force and inducing tensile forces in the legs. The force required to extract the leg is affected by several factors, including the nature of the soils, the depth of penetration, the geometry of the spudcan and whether soil backfill has occurred. The force available for leg extraction is frequently less than the force applied during installation. Where significant leg penetrations are attained, it is not uncommon for pulling of the legs to take several days, or in some cases much longer.

Where leg extraction problems are predicted, a warning should be included in the site-specific assessment report.

Potential mitigations include jetting and/or excavation of the surface soils. However, these measures can alter soil strength and the seabed topography, which can affect the future emplacement of jack-ups at the same site.

Further details can be found in References [A.9.4-24] to [A.9.4-29].

A.9.4.6 Cyclic mobility

General guidance on the assessment of the potential for liquefaction and/or cyclic mobility is given by Kramer^[A.9.4-30] and Idriss and Boulanger^[A.9.4-31]. Dean^[A.9.4-32] presents approximate methods for estimating settlements of submerged foundations subjected to time dependent loading.

A.9.4.7 Scour

The key conditions for scour are

- hydrodynamic conditions;
- flow disturbance due to presence of an obstruction;
- 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.

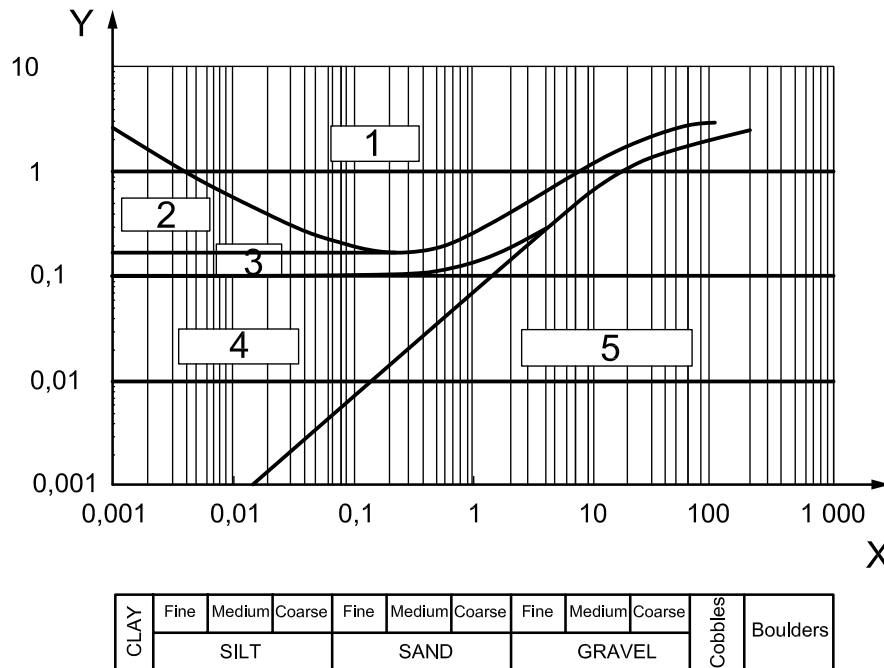
Particle size has a strong influence on the erodibility; see Figure A.9.4-1. Particle sizes larger than those of the original sea floor, such as gravels and cobbles can be useful for scour protection.

Scour is more important for spudcans with limited sea floor penetration, as removal of the soil can result in the following:

- a redistribution of leg forces or loss of jack-up hull trim;
- a reduction of the bearing capacity of the foundation and seabed fixity;
- eccentricity in the spudcan reaction;
- an increase in an existing potential for punch-through in layered soils.

There is no definitive procedure for the evaluation of scour potential, but useful reference material can be found in Sweeney *et al.*^[A.9.4-33]; Whitehouse^[A.9.4-34] and Rudolph *et al.*^[A.9.4-35]. Previous operational experience can help in the management of scour, either in the development of scour protection measures or of an awareness of the critical combination of tidal and non-tidal (storm driven) currents that can induce scour. Scour protection measures include the following:

- a) gravel dumping prior to installation, provided the selected gravel gradation does not cause damage to the jack-up spudcans: Particularly for the larger materials, care should be taken to ensure that this activity does not adversely affect future jack-up emplacements;
- b) use of frond mats, gravel bags, gravel dumping or grout mattresses after installation, the effectiveness of which can be evaluated from scour surveillance monitoring;
- c) monitoring and adjusting for reduction in hull elevation.

**Key**

- X particle size, expressed in millimetres
 Y mean flow velocity, expressed in millimetres per second
- 1 erosion
 2 transport/erosion
 3 transport
 4 sedimentation/transport
 5 sedimentation

Figure A.9.4-1 — Soil particle size and seabed mobility
 (after McDowell and O'Connor^[A.9.4-36])

A.9.4.8 Spudcan interaction with adjacent infrastructure

The interaction of the spudcans with adjacent infrastructure can be addressed with reference to the literature, e.g. Siciliano *et al.*^[A.9.4-37], Stewart^[A.9.4-38], Leung *et al.*^[A.9.4-39], and Kellezi *et al.*^[A.9.4-40].

A.9.4.9 Geohazards

Certain areas of the world, including the US, require shallow geohazard surveys and publish documents that can give some useful guidance, e.g. US MMS^[A.9.4-41] and OGP^[A.9.4-42]. 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.9.4.10 Carbonate material

No guidance is offered.

A.10 Structural response

A.10.1 Applicability

No guidance is offered.

A.10.2 General considerations

The ULS responses typically include overturning moments of the jack-up, reactions and displacements at the spudcans, horizontal deflections of the hull, the internal forces in the leg members and forces in the holding system. The responses should be obtained using appropriate combinations of functional actions, metocean or earthquake actions, and dynamic, second order and leg inclination effects with the action factors in Annex B. The application of actions is described in 8.8 and A.8.8. In 5.4.3, it is required that the analysis be carried out for a range of headings with respect to the jack-up such that the most onerous loading(s) for each item in the list above is/are determined.

When determining the FLS response, the cumulative number of stress cycles should be used to estimate the fatigue lives of steel components (see 10.6). Clause 10 is specifically aimed at short-term operations where fatigue is typically not a consideration. However, fatigue response can be important for long-term applications of a jack-up (see Clause 11).

A.10.3 Types of analyses and associated methods

The extreme storm ULS response can be determined either by a two-stage deterministic storm analysis procedure using a quasi-static analysis that includes an inertial loadset (see A.10.5.2) or by a more detailed fully integrated (random) dynamic analysis procedure that uses a stochastic storm analysis (see A.10.5.3).

Table 10.3-1 gives a list of some of the references used in an extreme storm response analysis. A common approach can be to start with a relatively simple analysis and to increase the level of complexity if the simple method shows the jack-up is unsuitable for the site.

Table A.10.3-1 — Cross references for extreme storm responses

Topic	Reference location	Comments and additional references
Metocean action calculation procedure	Table A.7.3-1	A.7 discusses actions, but Table A.7.3-1 gives an overview of the calculation procedure and gives references to the required input data, methods of calculating actions, and action factors.
Structural model	A.8	Table A.8.2-1 discusses the levels of detail in different structural models, and the information that can be obtained from them. A.8.3 to A.8.5 discuss modelling of the legs (including some simplified methods for calculating equivalent leg stiffness properties), the hull, and the leg to hull connection, respectively. A.8.7 discusses mass modelling.
Action factors	8.8	Action factors are given for both the two-stage, and one-stage stochastic storm analysis.
Application of actions	A.8.8	Wind and wave/current actions are determined through use of A.7.3. A.8.8 discusses application of actions, including functional actions, hull sagging, metocean actions, and inertial actions. Additional load cases that should be considered when $(T_n/T_p) > 0,9$, are given in A.10.5.2.2.3.
Large displacement effects	A.8.8.6	Different modelling techniques are discussed, including large displacement methods, geometric stiffness methods and negative springs.

Table A.10.3-1 (continued)

Topic	Reference location	Comments and additional references
Conductor actions	A.8.8.7	—
Damping	A.10.4.3	Table A.10.4-1 gives recommended explicit damping levels. A.7.3.3.2 describes relative velocity hydrodynamic damping and Equation (A7.3-15) gives the specific limits for when relative velocity formulation may be used. A.10.4.3.3 describes the hysteretic foundation damping that may be used in certain cases.
Two-stage deterministic storm analysis	A.10.5.2	In this method, a DAF is calculated and used to develop an inertial loadset that is combined with the maximum quasi-static wave action. The DAF can be from an SDOF analysis (A.10.5.2.2.2) or a random dynamic analysis (A.10.5.2.2.3). Figure A.10.4-2 gives an overview of a two-stage approach incorporating foundation fixity, which is normally included in the analysis. A.10.4.4.1.2 gives foundation modelling for a two-stage analysis.
SDOF analysis	A.10.5.2.2.2	This method is very simple and often used for a first pass assessment, but it has a limited range of applicability and, while normally conservative, can underestimate the DAF.
Random dynamic analysis	A.10.5.2.2.3	Commonly used to develop the dynamic response and then the DAF in a two-stage analysis Sets out the metocean and inertial loadset components of the basic load case that should be assessed for all values of (T_n/T_p) , and the extra load cases that should be considered when $(T_n/T_p) > 0,9$ Table A.7.3-3 gives specific recommendations on qualifying storm simulations. A.6.4.2.3 gives information on wave spreading using either three-dimensional analysis or a kinematics reduction factor. ISO 19901-1:2005, 8.3 and A.8.3, give information on the intrinsic and apparent wave periods, and the methodology for modifying the wave spectrum from intrinsic to apparent. A.10.5.3.3 gives additional details on all random wave dynamic analyses, regardless of whether it is for a one-stage or two-stage assessment. A.10.5.3.4 gives information on determining the MPME response, which is the result of the random analysis. Table A.10.5-1 gives recommendations for calculating the MPME and on the storm duration to use in the simulations.
Stochastic storm analysis	A.10.5.3	In this method, the MPMEs of the responses of interest (e.g. member utilizations) are determined directly in a one-stage analysis, although multiple one-stage dynamic analyses can be required (10.5.3). DAFs are not specifically developed. A.10.5.3.2 describes the determination and application of partial factors to the metocean parameters, as required in 10.5.3. Figure A.10.5-4 shows the analysis procedure for a one-stage stochastic storm analysis including foundation fixity. A.10.5.3.4 describes the determination of MPME responses.
Leg inclination	A.10.5.4	The effect of leg inclination is included in the structural code checks, but not in the global response analysis.

A.10.4 Common parameters

A.10.4.1 General

The ULS response can be calculated either by using a quasi-static analysis procedure including an inertial loadset or by using a more detailed (random) dynamic analysis procedure.

Clause 8 and A.8 identify the factors that affect the structural stiffness of the jack-up and discuss the structural stiffness modelling at various levels of complexity. The actions are discussed in Clause 7 and A.7.

The magnitude of the dynamic response is affected by the following:

- a) the dynamic characteristics (natural periods) of the structural system formed by the jack-up on its foundation;
- b) the characteristics of the excitation. For metocean excitation at sites with high current, there can be significant contributions from higher order harmonics in addition to those normally associated with quadratic drag terms and free surface effects.

The factors that affect these two characteristics are discussed in A.10.4.2 to A.10.4.5

A.10.4.2 Natural periods and affecting factors

A.10.4.2.1 General

The natural period of the jack-up on its foundation in the fundamental (or first) mode of vibration is an important indicator of the degree of dynamic response to be expected. The first and second vibrational modes are normally the surge and sway modes. The natural periods of these vibrational modes are usually close together; which of the two is the higher depends on which direction is less stiff. Where the natural or wave period varies with heading, care should be taken that the periods used are applicable to the direction being considered in the analysis. The third vibrational mode is normally a torsional mode, the three-dimensional effects of which can be important, in particular for headings where the legs and, hence, wave actions are not symmetric about the direction of wave propagation.

The natural period is dictated by the characteristics of the structural system, which are governed by the overall (global) structural stiffness, the mass and mass distribution, and the damping.

The undamped natural period is determined from Equation (A.10.4-1):

$$T_n = 2\pi\sqrt{M/K} \quad (\text{A.10.4-1})$$

where

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

M is the effective system mass;

K is the effective system stiffness.

ISO/TR 19905-2 contains a manual method for calculating the natural period. The method is not recommended for use in analyses but is useful for demonstrating some of the factors that affect the natural period of a jack-up.

A.10.4.2.2 Stiffness

The jack-up on its foundation represents a multi degree-of-freedom system. If available, a finite element structural model, containing the mass and stiffness properties of the jack-up should be used to obtain the

various natural periods and mode shapes. Structural modelling at various levels of complexity is discussed in A.8 and should consider stiffness contributions from the following:

- a) bending deformation of the legs;
- b) shear deformation of the legs;
- c) axial deformation of the legs;
- d) hull bending deformation;
- e) horizontal vertical and rotational leg-to-hull connection stiffness;
- f) horizontal, vertical and rotational foundation stiffness;
- g) second order P- Δ due to lateral displacement of the hull;

The model can contain a number of non-linear elements, notably the leg-to-hull connections and the spudcan-foundation interfaces.

If desired, the system stiffness for the fundamental modes can be determined from an idealized single degree-of-freedom system as described in ISO/TR 19905-2.

A.10.4.2.3 Mass

No guidance is offered.

A.10.4.2.4 Variability in natural period

No guidance is offered.

A.10.4.2.5 Cancellation and reinforcement

A.10.4.2.5.1 General

If the legs of the jack-up were lumped together at one position, waves passing through would cause each leg to have the same applied force history and the base shear transfer function (base shear versus wave period) would be a relatively smooth function. Assuming the leg kinematic parameters are axisymmetric, this transfer function would be the same for all wave headings. As the legs are moved apart, at an instant in time the wave position relative to each leg is different for each wave period. Since each leg is at a different phase for each wave period, the amplitude of the base shear transfer function at every period is bounded by the value with all legs together. Essentially, there is some force cancellation for almost all periods (smaller amplitudes than all legs together). Since the spacing between the legs changes by approach direction, different wave headings also result in different base shear transfer functions, even if the kinematic properties are still axisymmetric.

Figure A.10.4-1 shows cancellation and reinforcement periods. It can be used for a first evaluation of the position of the calculated natural period(s) relative to the cancellation and reinforcement points in the global loading. These can be characterized by the total horizontal wave loading or by the overturning moment; cancellation and reinforcement of points for these can occur at slightly different wave periods.

The assessor should aim to maximize the overall jack-up responses and not just, for example, the DAF.

The DAF calculated through the SDOF is independent of cancellation and reinforcement.

A.10.4.2.5.2 Quasi-static deterministic waves

Care should be taken to avoid cancellation in the quasi-static deterministic wave actions. This is not normally an issue; rarely is the extreme storm wave period close to a cancellation period, but if it is, a range of wave periods should be investigated (see A.6.4.2.9 and A.6.4.2.3).

A.10.4.2.5.3 Stochastic dynamic wave response

The natural period(s) used in the dynamic analysis should be selected such that a realistic but conservative value of the dynamic response is obtained for the particular application envisaged. Care should be taken to ensure that the response is maximized, not just the dynamic amplification, since it is possible to have a large DAF combined with low metocean excitation, due to cancellation, leading to low combined response. When the DAF is determined through a stochastic analysis, care should be taken to minimize cancellation (see also A.7.3.3.3.3) as this can result in significant underestimation of the DAF. In a two-stage stochastic analysis, the DAF is determined as the ratio of the responses of two models (see A.10.5.2.2.3): one that includes and one that excludes dynamic effects. A significant percentage of the dynamic effect is due to excitation of the natural period of the jack-up by that component of the wave trace having that same period. If there is cancellation at that period, there is little excitation, so the calculated DAF is unrealistically small.

Care should also be taken when there is significant current velocity as this can lead to slightly different cancellation effects. When combining current with a cyclic Morison wave loading, the drag term causes a harmonic excitation at half the wave period. This second harmonic can result in significant dynamic excitation, especially when the current is large and the period of the second harmonic is the same as the natural period. If cancellation of the second harmonic actions occurs, the DAF can be significantly underestimated.

In order to prevent cancellation resulting in potential underestimation of the DAF, the range of possible natural period(s) should be bracketed and compared with the relevant cancellation points in the global wave loading and the second harmonic of the wave period. When the natural period occurs at a cancellation point in the transfer functions, the mass or stiffness should be adjusted in a logical manner to move the natural period away from the cancellation point. The natural period should generally be increased above the cancellation point, by increasing the hull mass and reducing the foundation fixity, rather than reduced. This generally ensures that the dynamic response is maximized within reasonable limits.

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 for the case under consideration.

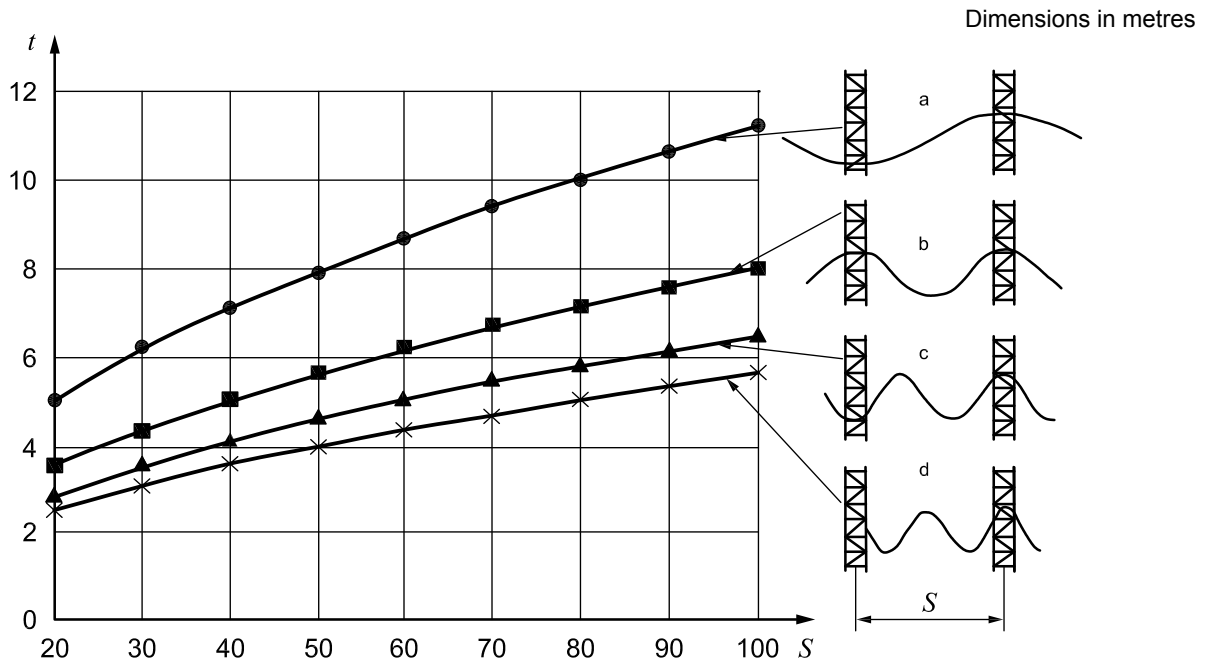
If the analysis is for pinned spudcans with maximum hull mass, then the adjustment should be made by reducing the hull mass (within the normal range) and/or by introducing a degree of rotational fixity at the seabed.

If the analysis is for a case with a degree of spudcan moment fixity, then the adjustment can most logically be made by varying the degree of rotational fixity at the seabed.

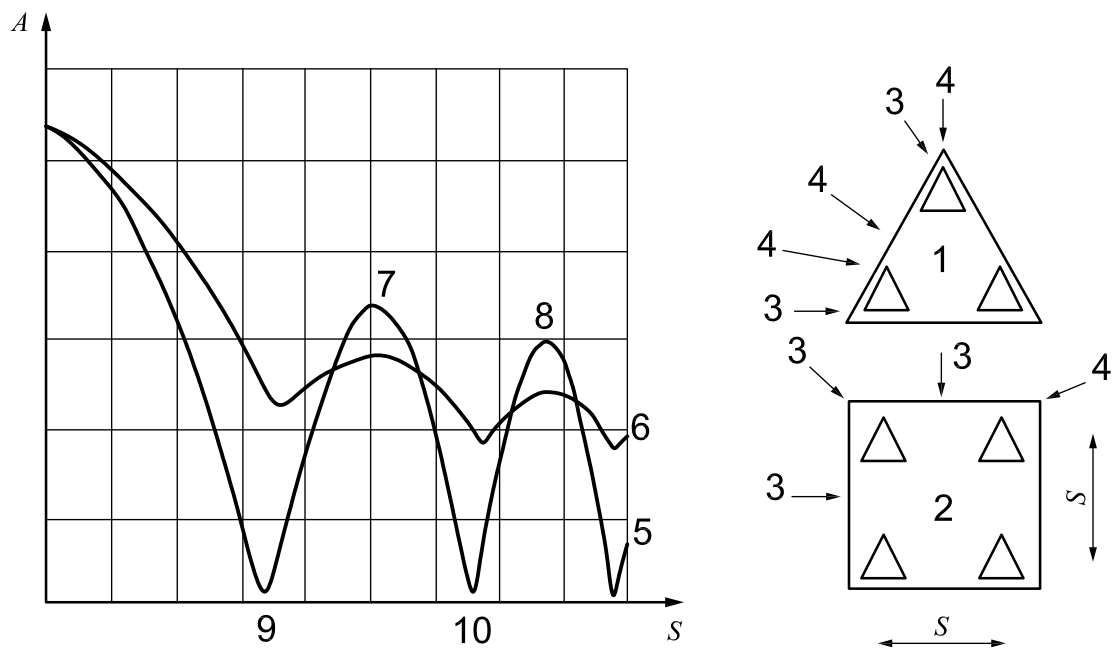
Alternatively, when the metocean data is omni-directional, the effects of wave spreading can be used to reduce the effects of cancellation by carrying out the dynamic analysis for a single wave heading along an axis which is neither parallel nor normal to a line through two adjacent leg centres. Thus, for a 3-legged jack-up with equilateral leg positions and a single bow leg, suitable analysis headings can be with the environment approaching from approximately 15° or 45° off the bow. The DAFs should be determined for one, or both, of these headings. The DAFs (or more conservative DAFs) can then be applied to the final quasi-static analysis for all headings.

In a one-stage stochastic analysis, similar care should be taken to avoid cancellation effects at both the natural period and at the predominant wave spectral energy.

Figure A.10.4-1 presents the periods at which first and second cancellations and reinforcements occur in the total wave actions. It is valid for the main wave directions of 3- and 4-legged jack-ups in water depths exceeding 30 m. The potential for increased response due to shortcrested waves should be considered (see A.7.3.3.3.3).

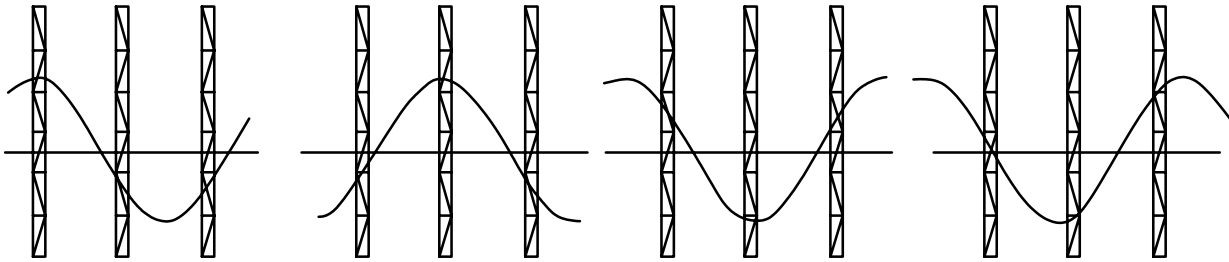


a) Sample of wave period in relation to wave force cancellation and reinforcement at all phase angles, including diagrammatic arrangement of jack-up legs with wave length



b) Horizontal action on jack-up versus wave frequency showing reinforcement and cancellation

Figure A.10.4-1 (continued)



c) Diagrammatic arrangement of legs on 3-legged jack-up in beam seas that can result in complete horizontal wave action cancellation at all wave phase angles

Key

- 1 3 legged jack-up
- 2 4 legged jack-up
- 3 wave direction versus leg locations associated with wave action curve 5
- 4 wave direction versus leg locations associated with wave action curve 6
- 5 indicative curve of wave action on jack-up versus frequency due waves in directions 3
- 6 indicative curve of wave action on jack-up versus frequency due waves in directions 4
- 7 first reinforcement point
- 8 second reinforcement point
- 9 first cancellation point
- 10 second cancellation point
- A* static wave action on jack-up
- f* wave frequency
- t* wave period
- S* jack-up leg spacing

- a First wave force cancellation over all wave phase angles; water depth ≥ 50 m.
- b First wave force reinforcement over all wave phase angles; water depth ≥ 30 m.
- c Second wave force cancellation over all wave phase angles; water depth ≥ 30 m.
- d Second wave force reinforcement over all wave phase angles; water depth ≥ 30 m.

NOTE 1 Figure A.10.4-1 a) has been drawn for effectively deepwater cases only. The reduced wave length in shallow water results in slightly longer wave periods producing first cancellation.

NOTE 2 On a 4-legged jack-up, it is possible to get complete cancellation of the horizontal actions at certain wave lengths (e.g. in a wave of specific length that results in two legs at the wave crest and two at the wave trough, as shown by line 'a' in Figure A.10.4-1 a). It is not possible to get complete cancellation of the horizontal actions on a 3-legged jack-up oriented with two legs parallel to the wave crest. There is partial cancellation in waves that result in one leg at a trough when two legs are at a crest, as shown by line 6 of Figure A.10.4-1 b), but there is not sufficient cancellation in any wave length to result in line 5. It can be possible to get complete cancellation on a 3-legged jack-up oriented with any two legs parallel to the direction of wave propagation, as shown in Figure A10.4-1 c), but it is not for precisely the wave periods given in Figure A.10.4-1 a).

Figure A.10.4-1 — Periods for wave force cancellation and reinforcement as a function of leg spacing

A.10.4.3 Damping

A.10.4.3.1 General

The main components of system damping are foundation, hydrodynamic and structural damping. Each of these can be modelled either linearly or non-linearly and can be calculated as part of the analysis or input as a percentage of critical damping (see Table A.10.4-1).

Structural damping is normally modelled linearly and input as a percentage of critical damping, however there are non-linear components (e.g. gaps in guides, pinion backlash).

Hydrodynamic damping is mainly due to fluid-structure relative velocity effects (see A.7.3.3.2); alternatively, a percentage of critical damping can be applied.

Foundation damping comprises three components: small strain material, hysteretic and radiation damping. The small-strain soil material damping is typically small. At larger strains, amplitude-dependent hysteretic damping can also occur. Where a non-linear foundation model is adopted for dynamic response analysis, the hysteretic foundation damping and soil stiffness reduction are accounted for directly. Where linearized soil stiffness is used in a time domain analysis, hysteretic damping should not be included.

A.10.4.3.2 Linear system damping

Where the model relies on damping defined as a percentage of critical, the total linear system damping should not exceed 7 % without credible, applicable justification. Lower values can be appropriate for fatigue analyses and lower sea states. Care should be taken to avoid the duplication of damping components when explicit and implicit representations are used simultaneously in the analyses. Table A.10.4-1 summarizes typical upper bounds when using percentages of critical damping.

Table A.10.4-1 — Recommended explicit damping from various sources

Damping source	Global linear damping not to exceed (% of critical damping)
Structure, holding system, etc.	2
Small strain foundation	2 ^a
Hydrodynamic	3 or 0 ^b

^a The small-strain soil material damping is typically small; in the absence of specific data, 2 % is considered to be a reasonable estimate.

^b In cases where the relative velocity formulation is used [$\alpha = 1$ in Equation (A.7.3-15)], the hydrodynamic damping is accounted for directly and should not be included as a percentage of critical damping.

A.10.4.3.3 Hysteretic damping

Foundation hysteretic damping can, in certain situations, increase the 2 % small-strain foundation damping given in Table A.10.4-1 and is discussed further in ISO/TR 19905-2.

A.10.4.3.4 Vertical radiation damping in earthquake analysis

In earthquake analyses, the foundation radiation damping from wave propagation can be included for vertical motion of the spudcan in addition to other foundation damping. Radiation damping should not normally be used in extreme storm or fatigue jack-up assessments. Radiation damping effects are implicitly included when the dynamic foundation analysis is performed using a continuum finite element analysis with a model that can accurately capture the effects of wave propagation in the foundation soils. Additional information on radiation damping is given in ISO/TR 19905-2. In simpler analyses, the vertical foundation radiation damping can be estimated from the work of Lysmer and Richart^[A.10.4-1], as given in Equation (A.10.4-2):

$$C_{rd} = R [0,85 B^2 / (1 - \nu)] \sqrt{(G_o \rho)} \quad (\text{A.10.4-2})$$

where

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

R is a reduction factor applied to avoid unconservatism, which should normally be taken as 0,5;

B is the equivalent spudcan diameter at uppermost part of bearing area in contact with the soil;

ν is Poisson's ratio of the foundation soil;

G_o is the shear modulus of the foundation soil [for clay, $G_o = G_{max}$, the maximum value of the shear modulus, that occurs at small strain (see A.9.3.4.3); for sand, $G_o = G$, the initial small-strain shear modulus (see A.9.3.4.4)];

ρ is the total, saturated, (mass) density of the foundation soil.

In non-linear dynamic analyses, or in linear time domain dynamic analyses using direct integration, Equation (A.10.4-2) can be used directly to establish the damping coefficients for the foundation dashpots.

In linear modal dynamic analyses, the additional contribution of vertical radiation damping to the linear damping ratio for the vertical mode only can be calculated as given in Equation (A.10.4-3):

$$\zeta_{rd} = R \cdot 0,232 \cdot B \cdot \omega_n \sqrt{(\rho/G_o)} \quad (\text{A.10.4-3})$$

where

ζ_{rd} is the radiation modal damping ratio to account for spudcan vertical motion;

ω_n is the angular natural frequency of the vertical mode, expressed in radians per second.

NOTE 1 The suggested value of 0,5 for R is a reduction on the amount of radiation damping and is comparable with values used in other industries. The reduction is intended to account for the frequency dependence and spatial variance (e.g., stratification) in soil conditions below the spudcan.

NOTE 2 Equation (A.10.4-2) is obtained by combining the definition of the damping coefficient, C , with the damping ratio of Equation (A.10.4-3) and the corresponding equation for stiffness given by Lysmer and Richart^[A.10.4-1].

NOTE 3 Radiation damping increases with increasing excitation frequency. Radiation damping levels from ocean wave excitation are expected to be less than 1 %, whereas for earthquake actions, radiation damping ratios can be large (>10 %). Radiation damping values this large can have significant effects on dynamic response.

A.10.4.4 Foundations

A.10.4.4.1 Foundations for extreme storm assessment

A.10.4.4.1.1 General

A.10.4.4.1 describes the analysis of the structure and the foundation evaluation which can be performed in two different ways:

- option 1: deterministic two-stage approach;
- option 2: stochastic one-stage approach.

A.10.4.4.1.2 Option 1 — Deterministic two-stage approach

Figure A.10.4-2 illustrates the procedure schematically.

In this approach, the dynamic response of the structure is evaluated based on either a simple linear analysis or a more complex elasto-plastic analysis in order to determine an inertial loadset. The dynamic analysis can include linearized springs. Typically, the initial linearized rotational stiffness for the dynamic analysis can be taken as 80 % of the value determined from A.9.3.4.1. This simplified approach does not capture the temporary reductions in stiffness that occur during plasticity events (generally with detrimental effects), but also does not capture the increased damping associated with these events (with beneficial effects).

The foundation and structural assessment is next performed using a quasi-static, iterative analysis technique, for which the dynamic actions have already been determined. This quasi-static analysis can be accomplished by means of either an elasto-plastic foundation model or by a simplified application of the full plasticity analysis as described below. This simple approach is used to apply moments on the spudcan by inclusion of a simple linear rotational spring. The moments thus applied are limited to a capacity based on the yield interaction relationship between the gross vertical force (F_V), the horizontal force (F_H) and the moment (F_M) acting on the spudcan.

This simple procedure is described in the following steps (see the right hand side of Figure A.10.4-2).

- a) Include vertical, horizontal and (initial) rotational stiffnesses (using linear springs, see A.9.3.4.1) in the analytical model and apply the factored functional and factored metocean actions together with the associated and separately calculated inertial loadset from a linearized dynamic analysis, to determine the resulting forces F_H , F_V and the moment F_M on each spudcan.
- b) Calculate the value of the yield interaction function (see A.9.3.3) using the resulting forces on each spudcan. If the value is zero, the force combination falls on the yield surface; for values greater than zero, it is outside; and for values smaller than zero, it is inside the yield surface.
- c) If a force combination initially falls within the yield surface, the rotational stiffness should be further checked to satisfy the reduced stiffness conditions in A.9.3.4.2.
- d) If the force combination initially falls outside the yield surface, the rotational stiffness should be arbitrarily reduced and the analysis should be repeated until the force combination at each spudcan lies essentially on the yield surface. If at that point the moment is reduced to zero and the force combination is still outside the yield surface, then a bearing failure (either vertical or horizontal) is indicated.
- e) Additional penetration due to a bearing failure can result in increased foundation capacity, which, in turn, expands the yield interaction surface. See A.9.3.3.5 and A.9.3.3.6 for guidance on expansion of yield interaction surface and A.9.3.6.6 for guidance on the displacement check.

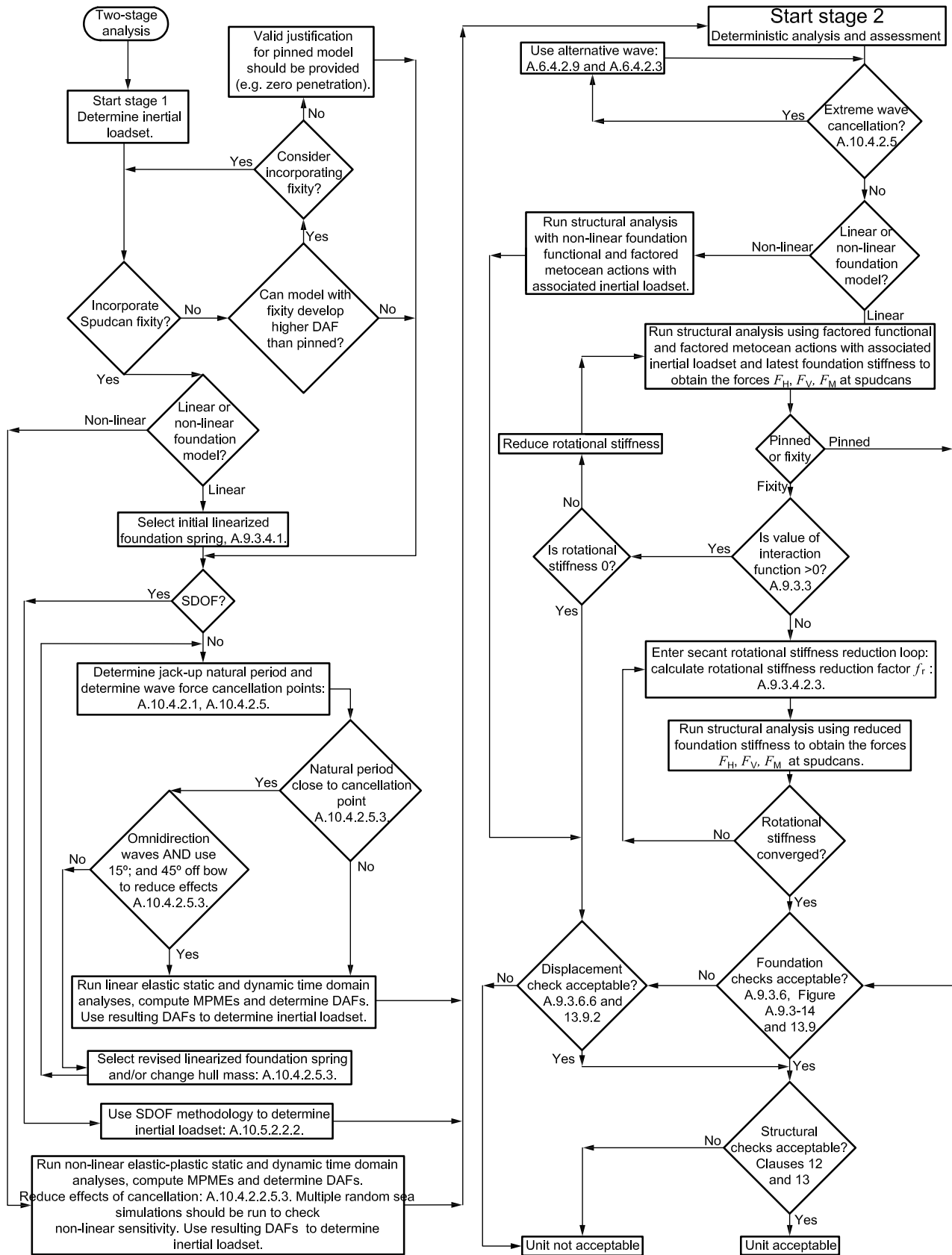


Figure A.10.4-2 — Analysis procedure for two-stage assessment with foundation fixity — Option 1

A.10.4.4.1.3 Option 2 — Stochastic one-stage approach

Figure A.10.5-4 illustrates the procedure schematically.

In this approach, the dynamic structural analysis and assessment is performed using one model. A fully detailed, non-linear time domain analysis is performed taking into account the elasto-plastic behaviour of the foundation.

The effects of the foundation fixity on the dynamic response and on the foundation reactions are simultaneously considered. This approach is more complete and often requires a complex incremental and iterative calculation procedure. The following outline procedure can be used.

- a) Use a time domain random dynamic analysis to determine structural response and foundation forces at each time step.
- b) Determine the foundation behaviour using a non-linear elasto-plastic model, such that at each time step the plastic and elastic portions of the behaviour are captured. If desired, this model can include hysteresis. This is likely to require an iterative procedure.
- c) As the dynamic response is influenced by the time history of the actions, a number of random dynamic analyses should be performed for differing input wave histories, and the MPMEs determined from a procedure described in A.10.5.3.4.

If, due to wave force cancellation effects, small changes in foundation stiffness result in significant changes in the response, the foundation stiffness should be selected with care to maximize the response (see A.10.4.2).

A.10.4.4.2 Foundations for earthquake assessment

For the simple screening assessment, the foundation should be modelled with the maximum interpreted shear modulus from Clause 9, without degradation and with appropriate rate adjustments.

For more detailed assessments, a fully non-linear coupled yield interaction model or a continuum model should be used with degradation effects.

A.10.4.5 Storm excitation

Currents change slowly compared with the natural periods at which jack-ups oscillate and can be considered to be a steady phenomenon. Variations in wind velocity cover a wide range of periods, but the main wind energy is associated with periods that are considerably longer than the natural periods of jack-up oscillations. Therefore, the wind can generally also be represented as a steady flow of air. The periods of waves typically lie between 3 s and 20 s. Since natural periods of jack-up in typical applications fall within this range, the primary source of dynamic excitation is from waves.

Sea waves are not regular but random in nature, with a more predominant periodicity when a swell is present. This has important implications that should be considered for both the dynamic excitation and the resulting dynamic response.

A.10.5 Storm analysis

A.10.5.1 General

No guidance is offered.

A.10.5.2 Two-stage deterministic storm analysis

A.10.5.2.1 General

In the first stage, an inertial loadset is determined from a dynamic amplification factor using either a single degree-of-freedom analogy ($K_{DAF,SDOF}$), see A.10.5.2.2, or a random wave time domain random dynamic analysis ($K_{DAF,RANDOM}$), see A.10.5.2.2.3. In the second stage, the maximum quasi-static wave/current action is determined by stepping the maximum wave through the structure. The maximum wave/current action is then combined with the inertial loadset to determine the responses. The maximum wave is defined in 6.4 and the methodology for calculating the quasi-static wave/current actions is described in 7.3. Load cases and combinations are discussed in 8.8.

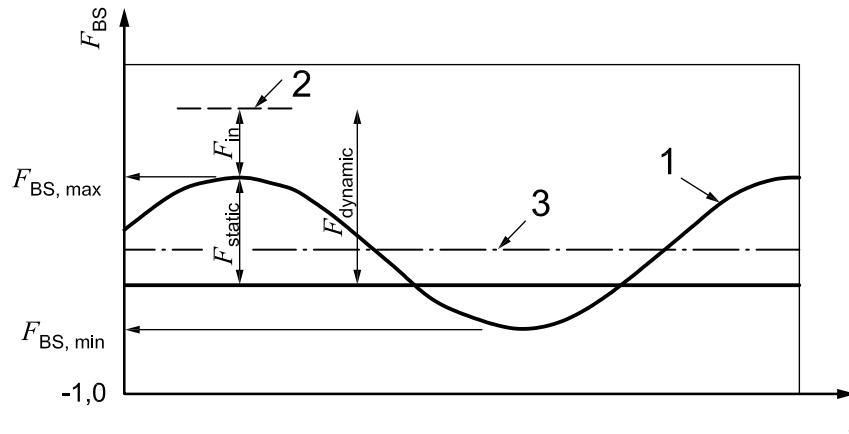
The spudcan-foundation interface can be modelled as described in 9.3.1.

A.10.5.2.2 Dynamic amplification factors (DAFs) and inertial loadsets

A.10.5.2.2.1 General

When using a deterministic analysis for calculating the jack-up's responses, the dynamic response is represented by equivalent inertial actions as described in A.8.8.5. The inertial loadset can be derived from the classical SDOF analogy described in A.10.5.2.2.2, or from the more complex random dynamic analysis method discussed in A.10.5.2.2.3; see Figure A.10.5-1. It should be recognized that dynamic amplification is the result of inertial actions that are dominated by the hull mass. Therefore, amplifying the hydrodynamic actions is not a correct physical representation.

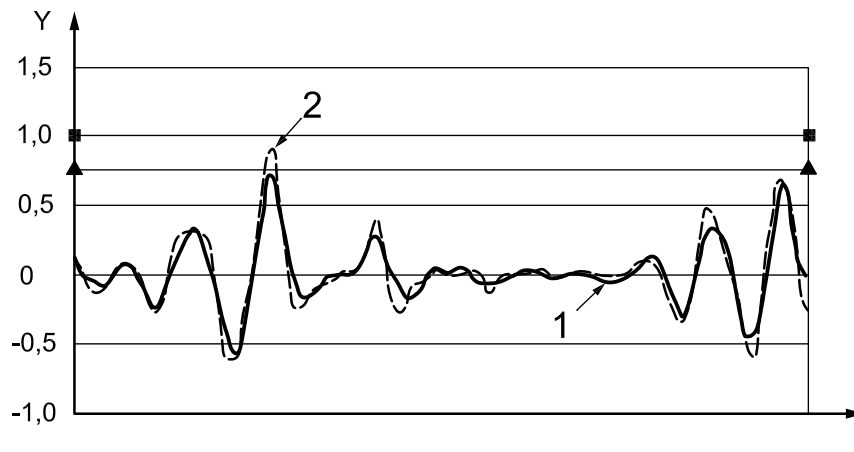
NOTE The difference between the height of the applied wave actions and the height of the system centre of mass means that the global response (e.g. base shear, overturning moment, hull deflection) and local response (e.g. member forces, holding system reactions, spudcan reactions) are not equally amplified by the inertial actions.



$$F_{in} = (K_{DAF,SDOF} - 1)[A_{BS(t)static}]$$

where $A_{BS(t)static}$, the amplitude of $BS(t)_{static}$, is equal to $0,5(BS_{max} - BS_{min})$

a) SDOF



$$K_{DAF,RANDOM} = R_{MPME,dynamic} / R_{MPME,static}$$

$$F_{in} = (K_{DAF,RANDOM} - 1) F_{static}$$

b) Stochastic/random

Key

t	time	BS	base shear
Y	normalized stochastic response	$F_{BS,min}$	minimum static base shear
1	static response	$F_{BS,max}$	maximum static base shear
2	dynamic response	F_{in}	magnitude of inertial loadset
3	mean of static response	$F_{dynamic}$	magnitude of total dynamic loadset
▲	$R_{MPME,static}$ stochastic response	F_{static}	magnitude of total static loadset
■	$R_{MPME,dynamic}$ stochastic response	R_{BS}	base shear response

Figure A.10.5-1 — Dynamic amplification factors

A.10.5.2.2.2 The classical SDOF analogy ($K_{DAF,SDOF}$)

This representation assumes that the jack-up on its foundation can be modelled as an equivalent single degree-of-freedom mass-spring-damper mechanism. The (highest) natural period of the jack-up's vibrational modes can be determined as described in A.10.4.2. The torsional mode and corresponding three-dimensional effects cannot be included in this representation.

The SDOF method is fundamentally empirical because

- the wave/current action does not occur at the hull;
- the excitation is non-periodic (random) and non-linear.

The method described below generally leads to an approximation of the jack-up's real behaviour that has been calibrated against more rigorous methods. The following cautions are noted when using the SDOF method.

- a) If the ratio of the jack-up natural period to the wave excitation period, Ω , is in the range 0,4 to 0,8 and the current velocity is small relative to the wave particle velocities, the SDOF method can give reasonable results, subject to items b) to d) below.
- b) The SDOF method does not account for reinforcement, as discussed in A.10.4.2.4, and this can make the method unconservative, particularly when $\Omega > 0,5$. When $\Omega > 0,5$, there can be significant energy in an irregular sea at the jack-up natural period, and this is not accounted for in the SDOF method because the DAF is not affected by any periodicity other than the excitation at $0,9T_p$. This lack of excitation is particularly important when the jack-up natural period is close to a wave reinforcement point. In this case, the resonant response, combined with reinforcement, can result in a significantly higher action than that calculated from the SDOF method. In the calculation of the natural period, a range in foundation fixity should be considered as this variability can shift the jack-up natural period within the base shear transfer function, resulting in different dynamic amplifications.
- c) The SDOF method can be unconservative for cases where the current velocity is large relative to the wave particle velocities. If the results of the assessment are close to the acceptance criteria, further detailed analysis is recommended.
- d) The SDOF method can be unconservative and should not normally be used in an extreme storm assessment when Ω is greater than 1,0, i.e. when $T_n > 0,9T_p$. However, the SDOF analogy may be used when the calculated Ω is greater than 1,0 providing Ω is taken as 1,0.

When using the SDOF method, a minimum value of 1,2 should be taken as the DAF in an extreme storm assessment, regardless of the DAF calculated using the SDOF method.

NOTE The DAF calculated in the SDOF analogy ($K_{DAF,SDOF}$) should not be directly compared to the DAF determined with a stochastic wave assessment ($K_{DAF,RANDOM}$). Because the method of determining the relevant inertial loadset is different, the same value of $K_{DAF,SDOF}$ and $K_{DAF,RANDOM}$ produce different total global responses; see Figure A.10.5-1.

The ratio of (the amplitudes of) dynamic to quasi-static response as a function of frequency (ω) or period (T) steady state, periodic and sinusoidal excitation is calculated by means of the classical dynamic amplification factor ($K_{DAF,SDOF}$) as given in Equation (A.10.5-1):

$$K_{DAF,SDOF} = \frac{1}{\sqrt{(1 - \Omega^2)^2 + (2\zeta\Omega)^2}} \geq 1,20 \tag{A.10.5-1}$$

where

Ω is the jack-up's natural period (T_n) divided by the wave excitation period (T_w); $\Omega = \frac{T_n}{T_w} \leq 1,0$;

ζ is the damping ratio or fraction of critical damping, $\zeta \leq 0,07$ (see A.10.4.3);

$$T_w = 0,9T_p;$$

T_p is the apparent peak wave period (modal or most probable period of the wave spectrum, corrected to account for current velocity; see A.7.3.3.5 and ISO 19901-1:2005, A.8.3);

T_n is the natural period as derived in A.10.4.2.1.

The damping parameter, ζ , in this model represents the total of all damping contributions (structural, hydrodynamic and soil damping). For the evaluation of extreme jack-up responses using the SDOF method, a value not exceeding 0,07 is recommended.

The calculated $K_{DAF,SDOF}$ from the SDOF analogy is used to estimate an inertial loadset, which represents the contribution of dynamics over and above the quasi-static response as illustrated in Figure A.10.5-1 a). The inertial loadset should be determined as given in Equation (A.10.5-2) and applied at the hull centre of gravity in the direction of wave propagation:

$$F_{in} = (K_{DAF,SDOF} - 1) F_{BS,Amplitude} \quad (A.10.5-2)$$

where

F_{in} is the magnitude of the inertial loadset;

$F_{BS,Amplitude}$ is the single amplitude of quasi-static base shear over one wave cycle,

$$F_{BS,Amplitude} = [F_{BS,(QS)Max} - F_{BS,(QS)Min}]/2;$$

$F_{BS,(QS)Max}$ is the maximum quasi-static wave/current base shear;

$F_{BS,(QS)Min}$ is the minimum quasi-static wave/current base shear.

Equation (A.10.5-2) is part of a calibrated procedure and should not be altered. A more general inertial loadset procedure, using the results from random dynamic analysis, is described in A.10.5.2.2.3.

A.10.5.2.2.3 Inertial loadset based on random dynamic analysis ($K_{DAF,RANDOM}$)

In the time domain random dynamic analysis procedure, two DAFs are calculated, one for the BS and one for the overturning moment (OTM). The inertial loadset, F_{in} , is calculated from these DAFs. The BS and OTM DAFs are the ratios of the MPME of the dynamic BS/OTM to the MPME of the static BS/OTM ($R_{MPME,dynamic}/R_{MPME,static}$), see Figure A.10.5-1 b), determined from corresponding dynamic and quasi-static time domain analyses for random-wave excitation according to the recommendations of the stochastic storm analysis in A.10.5.3. The MPME is defined in Table A.10.5-1.

Damping effects, including relative velocity effects, should not be included in the quasi-static (zero mass) analysis.

P- Δ effects should be included in both the quasi-static (zero mass) and the dynamic analyses. When P- Δ effects are included using negative springs, the same springs should be used in both analyses, although when calculating the BS DAF the shear force induced by the negative spring should be excluded. When the P- Δ effects are developed from gravity actions, the effects of vertical gravity loads should be modelled in the zero-mass analysis, i.e. weight is included even though there is no mass.

The inertial loadset, F_{in} , normally should be such that it increases both the BS and OTM from the deterministic quasi-static analysis by the same ratios as those determined between the random quasi-static (zero mass) analysis and the random dynamic analysis. In such cases, the structural model (used for dynamic analysis) may be simplified and it is not necessary that it contain all the structural details, but should nevertheless be a multi degree-of-freedom model. See A.8.8.5 for guidance on applying an inertial loadset to the model that matches both dynamic BS and OTM.

Caution should be exercised when the wave period approaches resonance and additional load cases should be considered when (T_n/T_p) is greater than 0,9. These extra load cases account for the changing phase between the forcing action and the inertial action as (T_n/T_p) approaches and exceeds 1,0 (see Figure A.10.5-3 and Note 1). The basic load case is the inertial loadset applied in phase with, and to increase the response to, the metocean actions, Equation (A.10.5-4). This load case is required for all ratios of (T_n/T_p) . Three additional load cases, Equations (A.10.5-5 to A.10.5-7), should be considered when (T_n/T_p) is greater than 0,9. Four sample load cases are shown diagrammatically in Figure A.10.5-3. In each case, the inertial loadset should be applied to the structure as described with A.8.8.5, using the same directional pair of $K_{DAF,RANDOM}$ values calculated for base shear and overturning moment.

NOTE 1 Figure A.10.5-3 shows the phase between the forcing action and the inertial action for an SDOF system for varying values of T_n/T_p and represents the underlying reason the extra load cases should be assessed in a two stage deterministic analysis when T_n/T_p is greater than 0,9. As the value of T_n/T_p increases beyond 0,9 the phase between the exciting action and the inertial action changes from being approximately in phase for low values of T_n/T_p , through being 90° out of phase when $T_n/T_p = 1,0$ to being approximately 180° out of phase when T_n/T_p is greater than 1,2. While Figure A.10.5-3 is drawn for an SDOF system, a similar phasing analogy can be made in a random dynamic analysis, albeit without the same degree of fine definition. It is because the phasing is not so well defined in a random seastate that extra cases should be considered when T_n/T_p is greater than 0,9.

The total base shear and overturning moment is the same in the first three load cases. Equations (A.10.5-4 to A.10.5-6) provide a match to the base shear but it is still necessary to correct the overturning moment. Both the base shear and overturning moment can be different in the fourth case: Equation (A.10.5-7); see Notes 5 and 6.

The base shear inertial loadsets are calculated as given in Equation (A.10.5-3):

$$F_{in,PHASE(a)} = K_{DAF,RANDOM} F_{STATIC} - F_{STATIC,PHASE(a)} \quad (A.10.5-3)$$

and are applied in load cases as given in Equations (A.10.5-4) to (A.10.5-7):

$$[E_e + \gamma_{f,D} D_e]_{(0)} = F_{WIND} + F_{STATIC} + \gamma_{f,D} F_{in,PHASE(0)} \quad (A.10.5-4)$$

$$[E_e + \gamma_{f,D} D_e]_{(90)} = F_{WIND} + \gamma_{f,D} F_{in,PHASE(90)} \quad (A.10.5-5)$$

$$[E_e + \gamma_{f,D} D_e]_{(180)} = F_{WIND} + F_{STATIC,UP} + \gamma_{f,D} F_{in,PHASE(180)} \quad (A.10.5-6)$$

$$[E_e + \gamma_{f,D} D_e]_{(-180)} = F_{WIND} + F_{STATIC} - \gamma_{f,D} F_{in,PHASE(-180)} \quad (A.10.5-7)$$

where

$[E_e + \gamma_{f,D} D_e]_{(a)}$ is the combined metocean actions and inertial actions for use as $(E_e + \gamma_{f,D} D_e)$ in Equation (8.8-1);

(a) is a subscript representing the notional phasing of the four different load cases given in Equations (A.10.5-4 to A.10.5-7) in which (a) is (0), (90), (180), and (-180), respectively (see Note 4);

F_{STATIC} is the deterministic quasi-static wave/current loadset in the direction of the MPME values;

F_{WIND} is the wind loadset;

$F_{STATIC,PHASE(a)}$ is the deterministic quasi-static wave/current loadset for the relevant load case:

it is equal to F_{STATIC} for the normal $PHASE_{(0)}$ case when used to calculate $F_{in,PHASE(0)}$ in Equation (A.10.5-4), which represents the normal case with inertia down-wind and crest wave loading [see Figure A.10.5-2 a)];

when $T_n/T_p > 0,9$:

it is equal to 0,0 for the PHASE₍₉₀₎ case when used to calculate $F_{in,PHASE(90)}$ in Equation (A.10.5-5), which represents the inertia only load case [see Figure A.10.5-2 b)];

it is equal to $F_{STATIC.UP}$ for the PHASE₍₁₈₀₎ case when used to calculate $F_{in,PHASE(180)}$ in Equation (A.10.5-6), which represents the case with inertia down-wind and trough wave loading [see Figure A.10.5-2 c)];

it is equal to $F_{STATIC.UP}$ for the PHASE₍₋₁₈₀₎ case when used to calculate $F_{in,PHASE(-180)}$ in Equation (A.10.5-7), which represents the case with inertia up-wind and crest wave loading [see Figure A.10.5-2 d)];

$F_{STATIC.UP}$ is the deterministic quasi-static wave/current loadset in the up-wind direction (i.e. maximum upwind loadset, which is normally in the opposite direction to the wind action).

Equations (A.10.5-4) to (A.10.5-7) represent the metocean and dynamic components, E_e and D_e , in Equation (8.8-1). The gravity components G_F and G_V should also be included when developing the complete assessment load case F_d in Equation (8.8-1). The response analysis should include P- Δ and hull sagging effects and the effects of leg inclination should be taken into account (see 7.8).

NOTE 2 It is relatively unusual to undertake a jack-up assessment where T_n/T_p is greater than 1,0 but such situations do exist, e.g. in relatively benign conditions in deep water and with low spudcan fixity. Experience has shown that in some cases the introduction of additional spudcan fixity reduces the natural period to below the wave period and this action results in an increased DAF.

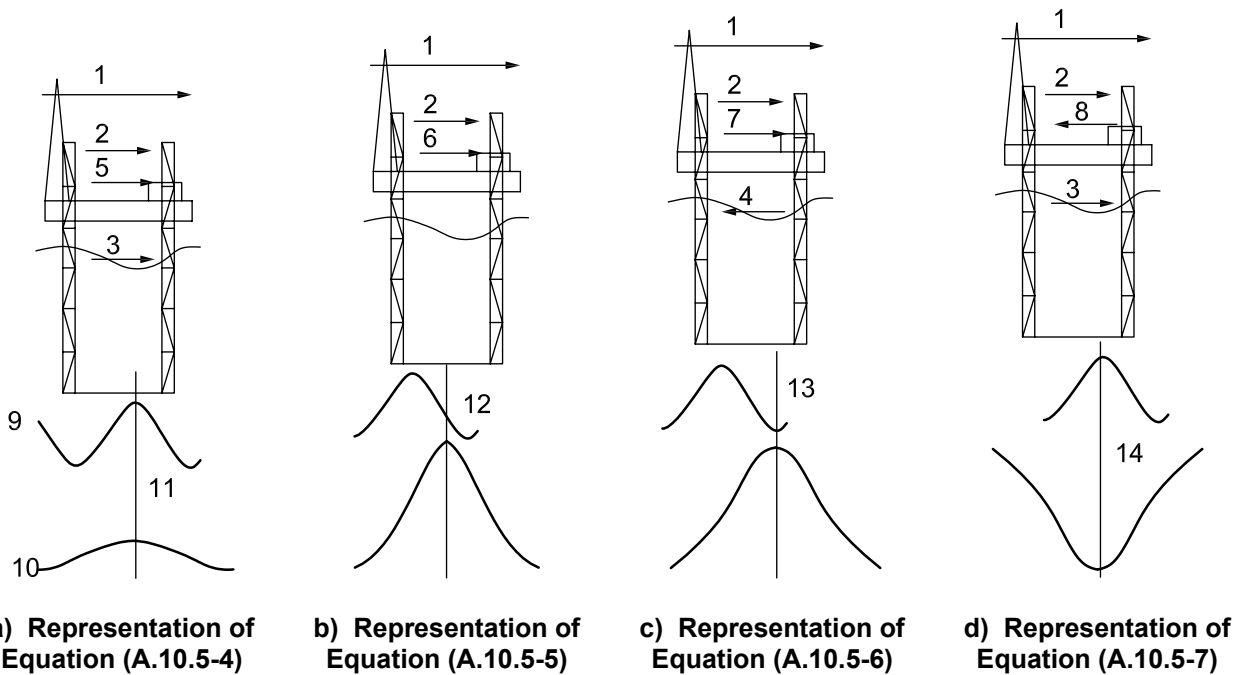
NOTE 3 Equation (A.10.5-3) is a scalar equation. It is used to determine the magnitude of the inertial loadset, but has no associated point of action or direction. Equations (A.10.5-4) to (A.10.5-7) are vector equations in which, for example, the inertial loadset is applied in the relevant direction and at the relevant elevation above the seabed.

NOTE 4 The subscript (a) is not the actual phase, but a notional, or indicative, phase taken from the SDOF analogy, similar to that given in Figure A.10.5-3, (in the full knowledge that the assessment is using a multi-degree of freedom model and loading). For example, PHASE₍₀₎ is not necessarily at the wave crest. It is simply used to represent the phase when the inertial loading is in-phase with the maximum down-wind wave/current action. Likewise PHASE₍₉₀₎ is used to represent the phase when the wave/current action is zero. PHASE₍₁₈₀₎ is used to represent the case when the inertia and direct wave/current actions are out of phase, with the inertial actions in the downwind direction and the wave/current actions, represented by the wave trough actions, in the up-wind direction. PHASE₍₋₁₈₀₎ is the reverse; the inertial actions are up-wind and the maximum wave/current actions are down-wind.

NOTE 5 The total vectored sum of the actions and moments that comprise $(E_e + \gamma_{f,D} D_e)_{(a)}$ should be the same in Equations (A.10.5-4) to (A.10.5-6). In effect, the base shear and overturning moment are the same for all of the first three cases: Equations (A.10.5-4) to (A.10.5-6). This is because the load cases are designed to represent different interpretations of the same results from the random time domain dynamic analysis. Consider the results of such an analysis: the procedure for results interrogation should have been established to capture the maximum base shear and overturning moment. However, it is possible that the relationship between these two values is not known (i.e. the maximum base shear can be occurring at a different part of the storm than the maximum overturning moment). It is, however, known that the values of both items are maximized. MPMEs are then calculated by the method of choice, and $K_{DAF,RANDOM}$ values are calculated for base shear and overturning moment. These DAFs are well defined, but it is not necessarily known of what components they are comprised. The intent of Equations (A.10.5-4) to (A.10.5-6) is to present three different sets of actions that can result in the different maxima base shear and overturning moments. Be aware that large correcting moments are likely to be necessary in Equation (A.10.5-5), the inertia-only load case. In Equation (A.10.5-5), the point of application of the actions has effectively moved from being predominantly close to the waterline (due to wave/current) with a relatively small inertial component at the hull centre of gravity to having the predominant action applied at the hull centre of gravity. Given that the hull centre of gravity is significantly higher than the point of application of the wave/current action and the requirement to have a consistent base shear and overturning moment, the introduction of large correcting couples at the hull is likely to be necessary.

NOTE 6 As stated in Note 5, the base shear and overturning moments are the same in Equation (A.10.5-4) to (A.10.5-6), so there are unlikely to be significant differences in global jack-up response. The importance of the different load cases is the location of the actions and the components that comprise them. This can result in different member loads and stresses.

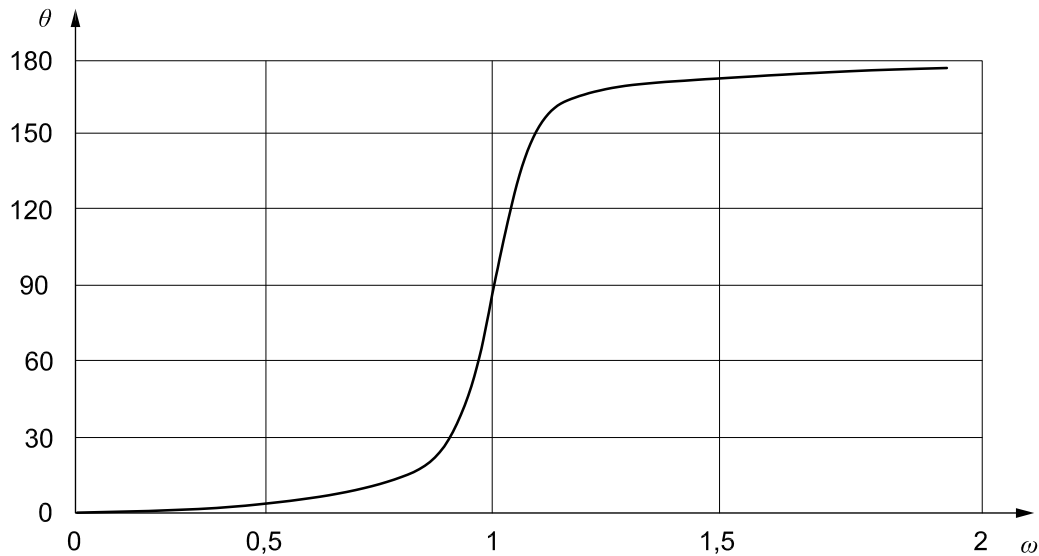
NOTE 7 Equation (A.10.5-7) can have a different combined base shear and overturning moment than Equations (A.10.5-4) to (A.10.5-6). In Equation (A.10.5-7), the magnitude of $\gamma_{f,D} F_{in,PHASE(-180)}$ is identical, for both base shear and overturning moment, to the value of $\gamma_{f,D} F_{in,PHASE(180)}$ in Equation (A.10.5-6), but it is applied in the opposite direction. This case represents the wave/current actions in the down-wind direction and the inertial actions in the up-wind direction. In most cases, the magnitude of the vector $(E_e + \gamma_{f,D} D_e)_{(-180)}$ is smaller than the magnitude of the equivalent vector in Equations (A.10.5-4) to (A.10.5-6). It is, however, possible that the internal leg stresses can be higher due to changes in internal leg shear and bending moments.



Key

- 1 direction of storm
- 2 wind action F_{WIND}
- 3 wave action F_{STATIC}
- 4 up-wind wave action at wave trough $F_{STATIC.UP}$
- 5 inertial loadset $F_{in,PHASE(0)}$
- 6 inertial loadset $F_{in,PHASE(90)}$
- 7 inertial loadset $F_{in,PHASE(180)}$
- 8 inertial loadset $F_{in,PHASE(-180)}$ with magnitude of base shear and overturning moment equal to $F_{in,PHASE(180)}$ but applied in the opposite direction
- 9 simplified representation of wave/current action on jack-up
- 10 simplified representation of inertial action on jack-up
- 11 line indicating relative phase of wave/current action and inertial action for $(a) = (0)$
- 12 line indicating relative phase of wave/current action and inertial action for $(a) = (90)$
- 13 line indicating relative phase of wave/current action and inertial action for $(a) = (180)$
- 14 line indicating relative phase of wave/current action and inertial action for $(a) = (-180)$

Figure A.10.5-2 — Diagrammatic representation of the load cases given in Equations (A.10.5-4) to (A.10.5-7) with the jack-up schematics showing the actions and the lower curves showing the phase between wave/current action and inertial action

**Key**

- ω ratio of the natural period, T_n , to the period of the forcing action, T_f
 θ phase angle in degrees between the forcing action and the inertial action

Figure A.10.5-3 — Phase between the forcing action and the inertial action for an SDOF system for varying ratios of natural period to forcing period (T_n/T_f) and damping of 7 % of critical

A.10.5.3 Stochastic storm analysis

A.10.5.3.1 General

In a stochastic storm analysis the extreme response can be predicted by stochastic methods where the intent is to determine the MPME of the responses of interest using statistical methods (see A.10.5.3.4). In the two-stage deterministic storm analysis, the MPMEs of the base shear and overturning moment are used to develop DAFs. For a one-stage stochastic storm analysis, the intent is to determine time histories of the utilizations from which the MPME utilizations can be calculated; see Figure 10.5-4.

In all stochastic analyses all action factors are set to 1,0 (see 8.8.1.3). When the stochastic storm analysis is used to determine a DAF (the first stage of a two-stage analysis), the metocean actions are unfactored in both the dynamic and the quasi-static analyses; the appropriate metocean action factor, $\gamma_{f,E}$, is applied in the second stage. However, when undertaking a fully integrated one-stage dynamic stochastic storm analysis that directly results in a time history of structural and foundation utilizations, the metocean parameters (i.e. wind velocity, wave height and current velocity) are factored; see A.10.5.3.2.

The waves can be modelled using a random superposition model, which is fully described in A.7.3.3.3.2, that identifies important constraints associated with this method of random wave dynamic analysis.

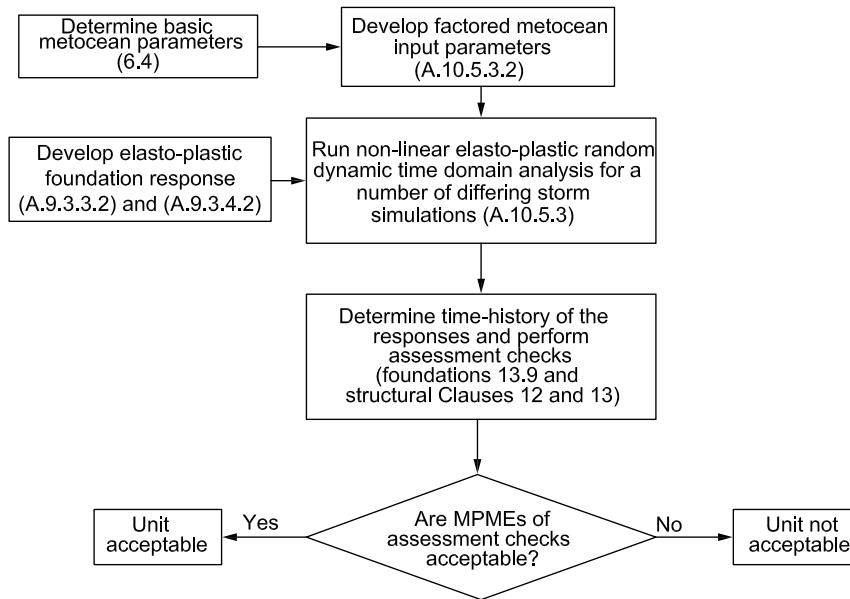


Figure A.10.5-4 — Analysis procedure for one-stage assessment with foundation fixity — Option 2

A.10.5.3.2 Application of partial factors to metocean parameters

When undertaking a one-stage fully integrated dynamic stochastic storm analysis, partial factors are applied to the metocean parameters. To ensure consistency between the one-stage stochastic and the two-stage deterministic approaches, the partial factors on metocean parameters should produce metocean action levels comparable to the factored quasi-static metocean actions used in the deterministic method.

When using dynamic stochastic storm analyses to determine a DAF for application in a two-stage deterministic analysis, the partial factors should be set to unity.

The partial factors on metocean parameters for fully integrated one-stage dynamic stochastic storm analyses can be determined as follows.

- Partial factor on wind velocity: The wind velocity used when generating the applied actions in accordance with A.7.3.4.1 should be factored by
 - $\sqrt{1,15}$ if 50 year return period independent metocean extreme storm actions are used, or
 - $\sqrt{1,25}$ if 100 year return period joint probability metocean data are used.
- Partial factors on wave height and current velocity: The partial factors for wave height and current velocity for use in the stochastic analysis are determined through an iterative process. The process involves factoring the wave height and current velocity until the metocean parameter-factored quasi-static stochastic wave/current action matches the action-factored quasi-static deterministic wave/current action computed using higher-order wave theory (see note below). The effects of wave spreading (see A.6.4.2.8) should be consistently included or consistently excluded in the stochastic and deterministic calculations used in the calibration. As a first approximation, the same partial factors can be used as given above for wind velocity. Some adjustment can be necessary to achieve a good or conservative match between the following two pairs of action values:
 - the stochastic MPME and the deterministic maximum, and
 - the stochastic mean and deterministic mean, the latter determined from integration over a full wave cycle (i.e. not from the average of the maximum and minimum values).

The match of MPME/maximum and mean actions is necessary to capture the cyclic behaviour. The adjustment generally results in different partial factors for the wave height and current velocity.

The wave period used in the stochastic analysis should be modified to maintain the same wave steepness as that of the unfactored sea state.

NOTE For the two-stage approach, the reference level for the wave and current actions is the quasi-static deterministic action. This reference level action is then modified through a DAF and the action factor to arrive at the final factored action. The important point is that the final action is founded on the quasi-static wave/current deterministic action. Conversely, in a fully integrated single stage analysis, there is no simple equivalent reference. It is, therefore, necessary to determine a stochastic equivalent to the factored deterministic quasi-static wave-current action. This is achieved by calculating the stochastic actions over three hours until partial metocean factors are found that match the MPME and mean actions with those from the action-factored quasi-static deterministic analysis. These partial metocean factors can, then, be used in the fully integrated stochastic dynamic analysis.

A.10.5.3.3 Random wave dynamic analysis method

Time domain simulations require that a suitable random sea state is generated, that the validity of the generated sea state is checked, and that the time step for the solution of the 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 Tables A.7.3-3 and A.10.5-1.

Wave spreading may be taken into account, either by using a three-dimensional analysis method or by using the kinematics reduction factor in a two-dimensional analysis (see A.6.4.2.3). Accounting for wave spreading generally results in a smaller DAF.

A.10.5.3.4 Methods for determining the MPME

The extreme response that should be checked in the assessment is the MPME response which has a 63 % chance of exceedance in a three-hour storm. This MPME response is defined in Table A.10.5-1 as the mode value or highest point on the probability density function (PDF). The stochastic waves modelled using a random superposition model result in non-Gaussian responses.

Four methods for obtaining the MPME of the response are included in Table A.10.5-1. Considerable care should be taken when T_n/T_p is greater than 0,8 and the use of any method to determine the MPME response should be critically assessed. When $T_n/T_p > 0,8$ other T_n/T_p ratios should be considered. The intent is to maximize the relevant responses (see A.10.4.2), but while not being unnecessarily conservative. This can be done by

- assessment of other wave height and period combinations (see A.6.4.2.9); or
- including or changing the level of spudcan fixity.

For the two-stage random dynamic analysis procedure the ratio of MPMEs of the dynamic to the quasi-static BS and OTM are used to determine the DAFs that are used to calculate the inertial loadset (see A.10.5.2.2.3).

Table A.10.5-1 — Recommendations for determining MPME

Method	Recommendations
General	The MPME is defined as the extreme with a 63 % chance of exceedance (typically this is the mode or highest point on the PDF). This is approximately equivalent to the 1/1000 highest peak level in a three-hour storm and the extreme with approximately a 63 % chance of exceedance.
Determination of the MPME from time domain simulations	Fit a Weibull distribution to the distribution of response maxima and determine the maximum value for the probability level of one exceedance in 3 hours. Take results as the average of MPMEs from at least 5 simulations. Each input wave simulation should be of sufficient length (usually more than 60 min, see Table A.7.3-3). See C.2.1. or Use multiple three-hour simulations and fit a Gumbel distribution to the absolute maximum from each simulation. Sufficient simulations (usually 10 or more) should be used to obtain stable MPME response values. See C.2.2. or Use Winterstein's Hermite polynomial model; when the kurtosis is > 5 use the improvements proposed by Jensen. Simulations of sufficient duration to provide stable skewness and kurtosis of responses (normally in excess of several hours). See C.2.3. or Use the drag-inertia method with appropriate scaling based on period ratio, to determine the DAFs for use in a two-stage deterministic storm analysis. Simulations of sufficient duration to obtain stable standard deviation of responses are required (usually more than 60 min). See C.2.4.
NOTE See C.2.	

A.10.5.4 Initial leg Inclination

The effects of initial leg inclination should be considered. Leg inclination can occur due to leg-to-hull clearances and hull inclination. Generally, hull inclination limits are set in the operations manual. The total horizontal offset due to leg inclination, O_T , can be estimated as given in Equation (A.10.5-8):

$$O_T = O_1 + O_2 \tag{A.10.5-8}$$

where

O_T is the total horizontal offset of the leg base with respect to the hull;

O_1 is the offset due to leg-to-hull clearances;

O_2 is the offset due to maximum hull inclination permitted by the operating manual.

If detailed information is not available, O_T should be taken as 0,5 % of the leg length below the lower guide.

It is necessary to account for the effects of leg inclination only in structural strength checks. This can be accomplished by increasing the effective moment in the leg at the lower guide by an amount equal to the offset O_T times the factored vertical reaction at the leg base due to fixed, variable, environmental, inertial and P-Δ actions.

A.10.5.5 Limit state checks

The ULS responses for assessment should be determined using appropriate combinations of actions due to fixed and variable load, wave/current actions and wind actions as required by the acceptance criteria in Clause 13. The application of actions is described in 8.8; 5.4.3 requires that the analysis be carried out for a range of headings with respect to the jack-up such that the most onerous force(s) for each item listed in

Table A.10.5-2 is(are) determined. The relevant ULS response parameters (action effects) are indicated in Table A.10.5-2.

Table A.10.5-2 — Action effects for limit state checks

Limit state check	Clause/Subclause	Response parameters(s) ^a	Action effect				
			G_F	G_V^b		E_e	D_e
				min	max		
Strength of members	12, 13.3	Member force vectors ^c	Y	Y ^d	Y	Y	Y
Spudcan strength	13.4	Forces on the spudcan	Y		Y	Y	Y
Holding system	13.5	Holding system force vectors	Y	Y ^d	Y	Y	Y
Overturning stability	13.8	Overturning moment				Y	Y
		Stabilizing moment	Y ^e	Y ^{d,e}			
Foundation capacity:	13.9, 9.3.6						
— preload	A.9.3.6.2	Vertical leg reaction	Y		Y	Y	Y
— sliding							
— pinned	A.9.3.6.3 or A.9.3.6.4.2	Vertical and horizontal leg reactions	Y	Y		Y	Y
— with moment fixity	A.9.3.6.5 (A.9.6.3.4.2)	Vertical, horizontal (and moment) leg reactions	Y	Y		Y	Y
— bearing							
— pinned	A.9.3.6.4.1	Vertical and horizontal leg reactions	Y		Y	Y	Y
— with moment fixity	A.9.3.6.5	Vertical, horizontal (and moment) leg reactions	Y		Y	Y	Y
— displacement	A.9.3.6.6	Spudcan displacements and reactions	Y	Y ^f	Y ^f	Y	Y

G_F = actions due to the fixed load positioned such as to adequately represent the vertical and horizontal distribution
 G_V = actions due to maximum or minimum variable load, as appropriate, positioned at the most onerous centre of gravity location applicable to the configurations under consideration
 E_e = metocean action due to the extreme storm event
 D_e = equivalent set of inertial actions representing dynamic extreme storm effects

^a In all instances the responses are assessed including the effects of deformation under functional actions (hull sag) and large displacement (P-Δ) effects.
^b Placed at most onerous centre of gravity position.
^c The effects of leg inclination to be included, which may be added after global response analysis (see A.10.5.4).
^d Consider minimum variable load if this is more onerous.
^e Fixed and variable load are included in response calculation so that P-Δ effects are captured.
^f As appropriate for the case under consideration: maximum for bearing and minimum for sliding.

A.10.6 Fatigue analysis

For jack-up operations of relatively long duration, see Clause 11.

A.10.7 Earthquake analysis

A.10.7.1 General

The provisions in A.10.7 complement ISO 19901-2 by presenting special aspects of an earthquake assessment procedure for jack-ups. The general procedures in Clauses 6 to 10 with the associated guidance in Annex A remain valid where appropriate; more specific reference to earthquake situations is provided in 6.6, 7.7, 8.6.3, 8.8, 9.4 and 10.3.

The greatest structural threat to a jack-up subjected to an earthquake is likely to be associated with vertical excitations that result in uneven settlement of the spudcans, which can cause lateral instability of the jack-up.

In situations where the jack-up is working over a platform, the relative motions between the platform and jack-up should be evaluated. The relative motions can affect the conductor and should be considered.

NOTE In earthquake environments, it is necessary that the operational issues (e.g. setback, cantilever and substructure and drilling rig clamping, drilling equipment) be given special consideration to ensure that major hazards to personnel are mitigated.

A.10.7.2 Earthquake assessment procedure

ISO 19901-2 gives alternative procedures for determining earthquake actions and alternative methods for the evaluation of earthquake activity. The selection of the procedure and the method of evaluation depend on the seismic risk category (SRC). The SRC depends on the exposure level and seismic zone in which the jack-up is to be located and is given in ISO 19901-2. The effects of near-source excitation should be considered (see A.10.7.5).

The simplified ELE screening methodology is given below and steps a) and b) are summarized in Table A.10.7-1.

- a) Determine earthquake actions using either the simplified earthquake action procedure or the detailed earthquake action procedure specified in ISO 19901-2 to develop spectral response accelerations for a bedrock base. Use of the simplified procedure (maps) for the initial screening of jack-ups is encouraged.
- b) Evaluate earthquake activity and the associated response acceleration spectra for the assessment of a jack-up against excitation of its base by ground motions using either ISO maps, regional maps or a site-specific earthquake hazard analysis, as specified in ISO 19901-2. Since ISO map accelerations are for a 1 000 year return period on rock, adjust the spectral shape for the 1 000 year event as described in ISO 19901-2 at the spudcan depth as a function of site soil characteristics.
- c) Perform response spectrum analysis in accordance with A.10.7.3.
- d) Evaluate the performance of the jack-up using the ULS assessment procedures provided in Clause 13.
- e) If the jack-up does not pass the simplified procedure, proceed to a more detailed assessment in accordance with A.10.7.4 using alternative analysis methods (10.9) and the ISO 19901-2 ALE procedures.

Table A.10.7-1 — Simplified procedure to develop 1 000 year ELE screening spectra using ISO 19901-2

Item	Source in ISO 19901-2
1 000 year accelerations – determine $S_{a,map}$	Annex B or site-specific
Determine the site seismic zone	6.4a
Simplified seismic action procedure	Clause 7
Determine soil site class	7.1a and Table 5
Spectral parameters C_a and C_v	7.1b and Tables 6 and 7
Develop horizontal spectrum	7.1c and Figure 2
Develop vertical spectrum	7.1d
Select damping	7.1e – use 5 % unless an alternate value justified
Seismic action procedure	7.2, $N_{ALE} = 1$ and $C_{r=1}$

A.10.7.3 ELE assessment

A.10.7.3.1 Partial action factors

The foundation, leg members and leg-to-hull connection should be assessed for the factored assessment actions defined in 8.8.1 for earthquake situations. The inertial action induced by the ELE ground motions should be determined using dynamic analysis procedures such as response spectrum analysis or time history analysis.

NOTE Reference can be made to Annex B, which contains all of the applicable partial action and resistance factors for a site-specific analysis.

Spudcan sliding should be considered for the minimum vertical reaction (uplift case) when the earthquake actions oppose the weight.

A.10.7.3.2 Structural and foundation modelling

The mass used in the dynamic analysis should consist of the mass of the structure associated with

- the fixed load G_F ,
- the best estimate of the variable actions; in lieu of specific data, 75 % of the maximum variable load G_V can be used,
- the mass of entrapped water, and
- the added mass.

The added mass can be estimated as the mass of the displaced water for motion transverse to the longitudinal axis of individual structural members and appurtenances (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. In addition, for the primary vertical mode, radiation damping according to A.10.4.3.4 can be included in the vertical response spectra definition. Additional damping, including hydrodynamic or soil induced damping (hysteretic and radiation), should be substantiated by special studies. Soil springs derived from small strain initial stiffnesses 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 non-linear stiffness and capacity of the foundation should be addressed in a manner compatible with Clause 9. If uplift or sliding is indicated from the screening analysis, non-linear dynamic time history or pushover analyses can be used to evaluate cumulative displacements and the resulting structural condition.

Vertical actions on the foundations should not normally exceed the preload. If the vertical actions on the foundations exceed the preload and the ULS Step 3 displacement check (see A.9.3.6.6) reveals the potential for excessive additional penetration, non-linear dynamic time history analyses with cyclic degradation can be used to evaluate cumulative displacements and the resulting structural condition, e.g. encroachment on an adjacent fixed platform.

A.10.7.4 ALE assessment

For jack-ups that do not satisfy the ULS criteria for the ELE screening assessment, a site-specific non-linear ALE assessment can be used to try and demonstrate acceptability. This can be satisfied by a pushover analysis or by time history analyses using ALE excitation.

Where substantial spudcan settlement or liquefaction is a possibility, a fully non-linear cyclic degrading analysis using best available soils modelling technology is recommended.

A.10.7.5 Near-source excitation

If operating close to an active fault (typically within about 15 km), it can be necessary to consider near-source ground motions. At these near-source distances, the ground motions can exhibit substantial rupture directivity effects and directionality, with motion characteristics often considerably in excess of normal design values, including permanent offsets, larger amplitude ground motions at relatively longer periods (e.g. $T \geq 1$ s), and vertical motions equal to or greater than horizontal motions at shorter periods (e.g. $T \leq 0,3$ s).

A.10.8 Accidental situations

No guidance is offered.

A.10.9 Alternative analysis methods

A.10.9.1 Ultimate strength analysis

No guidance is offered.

A.10.9.2 Types of analysis

When using the provisions of ISO 19902:2007, 7.10 (reserve strength analysis), care should be taken in modelling non-linear behaviour of chords and holding system of a jack-up structure.

A.11 Long-term applications

A.11.1 Applicability

No guidance is offered.

A.11.2 Assessment data

A.11.2.1 Jack-up data

A list of relevant modifications should be compiled including information about weights, wind areas and appurtenances added or removed that affect mass, applied actions and structural integrity.

For a long-term application, such modifications can typically include

- increased weight and wind area from such items as production modules, risers, flare towers, accommodation blocks, and conductors;
- increased wave and current actions due to risers, conductors or other structures exposed to waves.

A.11.2.2 Metocean data

The data required for a fatigue analysis should include long-term wave data in the form of a wave scatter diagram or a table of representative seastates, refer to A.6.4.2.10.

Joint probability and/or directional metocean data can be used to optimize the ULS and FLS assessment for the long-term application.

A.11.2.3 Geotechnical data

Effects of seabed scour, differential settlement, consolidation settlement, expected reservoir subsidence, sand waves, etc. can be of greater significance for long-term applications. For this reason, the site-specific geotechnical data should include the information necessary to evaluate these phenomena.

A.11.2.4 Other data

Further data associated with the long-term application can be required. Examples include the possible effect on geotechnical properties due to top-hole construction activities, marine growth, effects from adjacent structures, etc.

A.11.3 Special requirements

A.11.3.1 Fatigue assessment

A.11.3.1.1 Historical damage

The assessment should take into account the fatigue history of critical details prior to installation on the planned site and focus on details of member connections that are essential to the overall structural integrity of the jack-up. In order to assess existing fatigue damage, specific information relevant to prior installations is required. The availability of the information depends on the information collected and retained by the jack-up owner over the life of the jack-up. The quality of the database affects the historical results. The historical data can have a large variability, requiring the assessor to make assumptions in the historical fatigue assessment. The assessment can include detailed fatigue analysis of the historical data and/or evaluation of inspection records. Parameters identified as important in addressing the historical aspects of jack-up fatigue are as follows:

- geographic region (e.g. Gulf of Mexico, North Sea, Eastern Canada, etc.) and, where available, the coordinates of the previous sites so that metocean parameters can be developed for use in historical analysis;

- hull elevation and orientation;
- water depth;
- penetration;
- soil type and characteristics.

A.11.3.1.2 Fatigue sensitive areas

Areas that are susceptible to fatigue damage include

- leg members and joints in the vicinity of the upper and lower guides for the operating leg/guide location;
- leg-to-hull holding system;
- leg members and joints adjacent to the waterline;
- leg members and joints in the lower part of the leg near the spudcan; and
- spudcan-to-leg connection.

Normally, it is not necessary that the fatigue assessment include consideration of the hull structure since the long-term cyclic loading is similar to that experienced in multiple short-term operations. Generally the hull is not fatigue sensitive.

A.11.3.1.3 Special considerations for fatigue assessment

Special considerations in the fatigue assessment are listed below.

- Inclusion of detailed models to arrive at local stress levels:

Areas in the structure with high stress levels can be identified using models developed for global analysis and the stress ranges determined using appropriate stress concentration factors (SCFs) from literature. Alternatively, more detailed fine-mesh finite element models can be used to determine the hotspot stress ranges (suitable methodologies are given in 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 for clay and A.9.3.4.4 for sand). Typically, the fatigue assessment of the spudcan and lower part of the leg requires the use of upper bound stiffness, while the fatigue assessment for the upper leg and the leg-to-hull interface requires lower bound stiffness. Although the foundation stiffness varies as a function of the reactions beneath the spudcan, the variation is unlikely to be of significance except, possibly, for low-cycle fatigue.

- Inclusion of non-linearities and dynamics:

The structural response of a jack-up is such that pure linear techniques can be inadequate. Therefore, the analysis should include the non-linear effects of the structure. These can include

- hydrodynamic actions,
- large displacement effects (see 8.8.6),
- dynamic amplification (see 10.5.2, 10.5.3),
- leg-to-hull interface, e.g. ensuring that those structures that transfer force in compression contact only are properly modelled.

A.11.3.1.4 Fatigue analysis methodology

A robust analysis method should be used to determine the fatigue damage. The method should determine the response of the jack-up structure to various sea states representing the operational environment. The jack-up should be considered in the operational configuration, which includes the levels of variable load, hull elevation and cantilever position.

Wave spreading and directionality effects can be included.

Foundation stiffnesses are generally assumed to be linear in smaller sea states. A check of non-linearity should be performed to validate this assumption for higher sea states.

For guidance on suitable fatigue analysis methodology, S-N curves and SCFs the assessor is referred to one of the integral methods outlined in Table A.11.3-1. These should be used taking account of the specific structural characteristics of the jack-up as described above.

For fatigue analysis the partial action factor should be reduced to unity when using S-N curves at the mean minus two standard deviations of $\log(N)$.

Table A.11.3-1 — Sources of guidance on fatigue analysis methodology

Organization	Document	Reference
DNV	Methods are given in DNV-OS-C104	[A.11.3-1]
	Technical guidance on fatigue calculations, e.g. calculation methods, SN-curves, SCFs are given in DNV-RP-C203 Fatigue Design of Offshore Steel Structures	[A.11.3-2]
ABS	Methods are given in the Guide for the Fatigue Assessment of Offshore Structures (April 2003)	[A.11.3-3]
	Commentary on the Guide for the Fatigue Assessment of Offshore Structures (April 2003)	[A.11.3-4]
API	Methods are given in RP2A-LRFD-1993	[A.11.3-5]
UK HSE	Guidance is given in OTO 2001/015 and	[A.11.3-6]
	OTH92 390	[A.11.3-7]
ISO	Methods are given in ISO 19902	—

A.11.3.1.5 Fatigue acceptance criteria

The fatigue analysis should determine the fatigue damage in the period before, as well as during the long-term application of the jack-up. The margin of safety of a structural detail depends on its accessibility for inspection and the availability of one or more alternative load paths (redundancy) after failure of the detail investigated. The acceptance criterion for fatigue strength is as given in Equation (A.11.3-1):

$$f_{FD,e}D_{c,e} + f_{FD,s}D_{c,s} < 1,0 \quad (\text{A.11.3-1})$$

where

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

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

$f_{FD,e}$ is the fatigue damage design factor applicable to $D_{c,e}$; generally, $f_{FD,e} = f_{FD,s}$, but $f_{FD,e}$ should not be taken larger than 2 if the detail has been inspected thoroughly before the long-term application;

$f_{FD,s}$ is the fatigue damage design factor applicable to $D_{c,s}$; see Table A.11.3-2 or A.11.3-3.

Table A.11.3-2 — Fatigue damage design factor $f_{FD,s}$

Fatigue damage design factor, $f_{FD,s}$	Full access for inspection and repair	Access for inspection, no repair during operation	No access for inspection, no repair during operation
Full redundancy/minor consequence	2	3	5
No redundancy/major consequence	3	5	10

The values in Table A.11.3-3 give more detailed guidance for structures that are fully redundant, i.e. the structure does not have single members or member connections that, when damaged, can cause a failure with major consequence. This is typical of RCS approved jack-ups with braced legs.

Table A.11.3-3 — Fatigue damage design factor $f_{FD,s}$ — Redundant structure

Description (assumes there is structural redundancy for every member and member connection)			Fatigue damage design factor, $f_{FD,s}$
Can inspect and repair	Hull structure	Primary hull structure	1
		Leg-to-hull interface structure with access for inspection and repair	2
	Leg structure in air	Leg chords, brace to chord joints, brace joints	2
Can inspect but not repair	Leg structure in splash zone	Leg chords, brace to chord joints, brace joints	3
	Leg structure under water	Leg chords, brace to chord joints, brace joints, leg to spudcan connection	3
	Spudcan	Structure with access for inspection and repair	3
Cannot inspect or repair	Hull structure	Leg-to-hull interface structure without access for inspection and repair	5
	Leg structure under sea floor	Leg chords, brace to chord joints, brace joints, leg to spudcan connection	5
	Spudcan	Structure without access for inspection and repair	5

If necessary, fatigue life enhancement methods such as weld profiling, weld toe grinding and peening may be used, subject to RCS approval. Peening should only be used for improving fatigue lives after appropriate inspection.

A.11.3.2 Weight control

A weight control procedure should be prepared by the party responsible for operating and maintaining the jack-up during the long-term application. The procedure should be used to track the changes in weights and to ensure ongoing 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 is offered.

A.11.3.4 Marine growth

Marine growth should be taken into account in the site-specific assessment. The assessment can be for either the growth specified for the application period or for a pre-determined limit. In either case, the actual growth should be monitored and, when necessary, removed to ensure compliance with the assessment assumptions.

A.11.3.5 Foundations

Settlements can occur (see 11.3.5 and A.11.2.3), resulting in the loss of air gap or the hull being out-of level. The consequences of resolving these should be considered in the assessment, e.g. the effect of guide position on the fatigue or strength analyses, changes in conductor support, etc.

Consolidation of the soil through dissipation of pore pressures during the long-term operation can result in changes in foundation strength and stiffness. This affects the redistribution of leg moments and changes the dynamic response. The effects on fatigue life and strength should be considered, especially at the leg to spudcan connection.

In conditions where scour can occur, scour protection can be required.

A.11.4 Survey requirements

A.11.4.1 Pre-deployment inspection plan

The RCS special survey requirements prior to a long-term application can be more extensive than those of a typical special survey. Therefore, it is advisable to plan the surveys prior to mobilisation to a shipyard for modifications. The inspection plan should specify the locations and types of inspection, taking into account the areas that the assessor has identified as being critically stressed during the extreme storm or being fatigue sensitive during the long-term application. Areas that are not accessible, or are difficult to access for in-service inspection, should be subject to more detailed pre-deployment inspection and should be specially evaluated (see A.11.3.1).

A.11.4.2 Project specific in-service inspection programme

The project specific in-service inspection programme (PSIIP) should be developed by modifying and updating the existing in-service inspection programme normally required by the RCS. The PSIIP should reflect the requirements for the planned long-term application.

NOTE The PSIIP is likely to be subject to direction and approval by the RCS.

Areas that require special inspection procedures, such as underwater parts, should have documented inspection procedures, giving due consideration to the most suitable and practical methods.

The results of the in-service inspections should be reviewed and, if appropriate, the PSIIP modified to reflect the results of this review. This information can be relevant to ensure the ongoing validity of the PSIIP and for extending the jack-up's time on site beyond that originally planned.

A.11.4.3 Alternative project specific in-service inspection programme (PSIIP)

An alternative can be derived using a probabilistic approach. The safety philosophy behind the alternative PSIIP should be in accordance with the RCS's safety philosophy and the structural reliability level inherent in the RCS rules should be maintained. The approach developed should be documented.

When using a probabilistic approach, it should be recognized that uncertainties are associated with prediction of the fatigue performance and the inspection techniques applied. Key uncertainties should be accounted for in the probabilistic analysis.

A.12 Structural strength

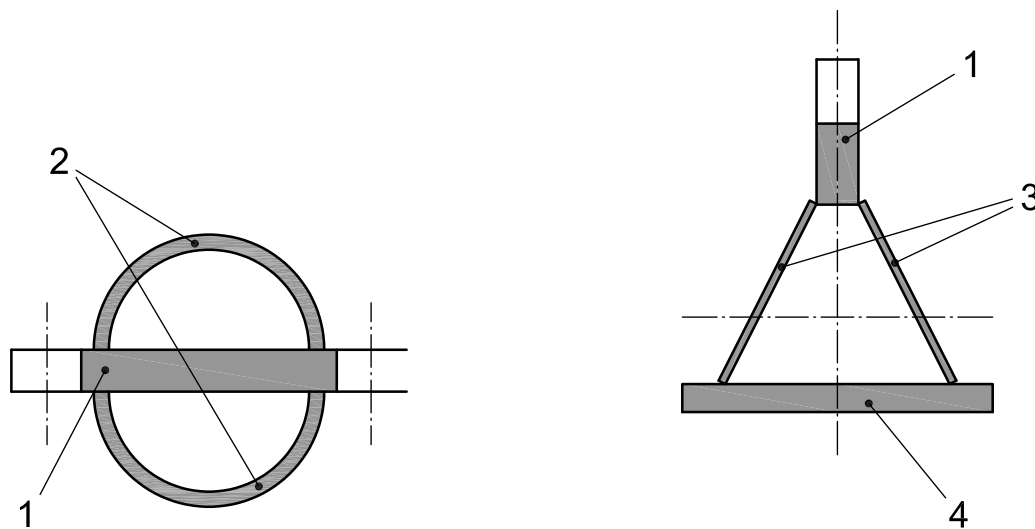
A.12.1 Applicability

A.12.1.1 General

A.12 applies to steel structures only. Where necessary, the equations included in A.12 have been non-dimensionalized using Young's modulus, E , of 205 000 N/mm² (or 29 700 ksi).

For the purposes of strength assessment, it is necessary to consider the truss type leg structure as being comprised of structural members. Typically, each structural member can be represented by a single beam-column element in an appropriate analytical model of the structure. Examples of structural members are braces and chords in truss type legs and box or tubular legs, all of which form a part of the structure for which the properties can readily be calculated.

The cross-section of a non-circular prismatic structural member is usually comprised of several structural components. Table A.12.2-1 shows classification limits for circular and non-circular prismatic members in typical jack-up chords comprising split-tubulars, rack plates, side plates and back plates (see Figure A.12.1-1). A component is by definition comprised of only one material. Therefore, where a plate component is reinforced by another piece of plating of a different yield strength (see Figure A.12.2-1) the reinforcing plate should be treated as a separate component. Non-circular prismatic members should be assessed using the provisions of A.12.6.



a) Opposed rack split tubular chord member section

b) Triangular type chord member section

Key

- 1 rack plate component
- 2 split tubular component
- 3 side plate component
- 4 back plate component

Figure A.12.1-1 —Typical components of typical jack-up chord cross-sections

Tubulars should be assessed as structural members using the provisions of A.12.5.

In Clause 12, subscripts y and z are used to define the two axes of bending of tubular and prismatic members, however F_y is used to define the yield strength in stress units.

NOTE The structural resistance factors for tubular members given in Clause 12 are based on an independent interpretation of the theoretical values derived from the data used in the calibration of API RP 2A LRFD, 1st edition, to API RP 2A, 15th edition, and the data used in the development of the ISO 19902 tubular members strength formulations. The values for non-tubular prismatic members were taken from AISI; see Reference [A.12.5-1], which changed its equivalent resistance factor from 1,18 to 1,1 between the 1986 and 2005 editions because of a reassessment of the applicable data, which resulted in an effective reduction in the coefficient of variation.

A.12.1.2 Truss type legs

No guidance is offered.

A.12.1.3 Other leg types

No guidance is offered.

A.12.1.4 Fixation system and/or elevating system

No guidance is offered.

A.12.1.5 Spudcan strength including connection to the leg

No guidance is offered.

A.12.1.6 Overview of the assessment procedure

No guidance is offered.

A.12.2 Classification of member cross-sections**A.12.2.1 Member type**

No guidance is offered.

A.12.2.2 Material yield strength

The value of the yield strength taken from a tensile test should correspond to the 0,2 % offset value. Where this value is greater than 90 % of the ultimate tensile strength (UTS), the yield strength, F_y , used in A.12 should be taken as 90 % of UTS. The following variables are used in A.12:

F_y is the yield strength in stress units (minimum of the yield strength and 90 % of the UTS);

F_{yi} is the yield strength of the i th component of the cross-section of a prismatic member, in stress units (minimum of the yield strength and 90 % of the UTS of the i th component of the cross-section);

F_{ymin} is the minimum yield strength of the F_{yi} of all components in the cross-section of a prismatic member, in stress units;

F_{yeff} is the effective yield strength of the cross-section of a prismatic member, in stress units, determined from the plastic tensile axial strength divided by the minimum cross-sectional area.

A.12.2.3 Classification definitions**A.12.2.3.1 Tubular member classification**

A cross-section of a tubular member is a class 1 section when Equation (A.12.2-1) applies:

$$D/t \leq 0,0517 E/F_y \quad (\text{A.12.2-1})$$

where

D is the outside diameter;

t is the wall thickness;

F_y is the yield strength in stress units;

E is Young's modulus of steel ($E = 205\,000 \text{ N/mm}^2$).

NOTE Compliance with class 1 classification is relevant only when undertaking earthquake, accidental or alternative strength analyses (see 10.7, 10.8 and 10.9). In all other cases, the distinction between class 1 (plastic) and class 2 (compact) is irrelevant to the assessment.

A.12.2.3.2 Non-circular prismatic member classification

Non-circular prismatic members that contain curved or tubular components should have the curved components classified based on the values given in Table A.12.2-1 and their flat components classified based on Tables A.12.2-2 to A.12.2-4. The limits given in Table A.12.2-1 tend to be conservative as, in most cases, there is additional support for the curved component by the flat components (e.g. the rack in a split tube chord reinforces the split tube and helps to prevent local buckling). When the limits given in Table A.12.2-1 are considered to be too onerous, it can be possible to justify the use of alternative limits through rational analysis.

NOTE The use of Tables A.12.2-3 and A.12.2-4 to classify cross-sections subject to axial compression and bending is complicated and requires knowledge of the cross-section stress distribution. It is always acceptable to conservatively base the cross-section classification on the relevant axial compressive case.

Table A.12.2-1 — Classification limits for non-circular prismatic members containing curved components

Class	<i>D/t</i> limits	
	Section in bending	Section in compression
1	$D/t \leq 0,052 E/F_y$	$D/t \leq 0,052 E/F_y$
2	$D/t \leq 0,103 E/F_y$	$D/t \leq 0,077 E/F_y$
3	$D/t \leq 0,220 E/F_y$	$D/t \leq 0,102 E/F_y$
4	$D/t > 0,220 E/F_y$	$D/t > 0,102 E/F_y$

When classifying non-circular prismatic components in accordance with Table A.12.2-2 to A.12.2-4, a distinction is made between internal components and outstand components as follows:

- a) internal components are components that are supported by other components along both longitudinal edges, i.e. the edges parallel to the direction of compression stress, and include
 - flange internal components: internal components parallel to the axis of bending,
 - web internal components: internal components perpendicular to the axis of bending;
- b) outstand components are components that are supported by other components along one longitudinal edge and at both ends of the member under consideration, with the other longitudinal edge free.

When a cross-section is composed of components of different classes, it is classified according to the highest (least favourable) class of its compression components. Slender components within a cross-section can be ignored, provided that only the remaining cross-section is used for all aspects of the assessment. However, if a 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 Tables A.12.2-2 to A.12.2-4, the dimensions that should be used are those given in the relevant table. The components are generally of constant thickness; for components that taper in thickness, the average thickness over the width of the component should be adopted.

Members that do not satisfy the applicable simplified lateral torsional buckling (LTB) criteria should be assessed further to determine a reduced representative member bending moment strength, M_b , using the guidance in A.12.6.2.6.

The LTB criterion for singly symmetric open sections is taken from F2-5 of AISI^[A.12.5-1], as given in Equation (A.12.2-2):

$$\frac{L_b}{r_{ltb}} \leq 1,76 \sqrt{\frac{E}{F_{y,ltb}}} \quad (\text{A.12.2-2})$$

The LTB criterion for any closed section is derived from BS 5400-3^[A.12.5-2], as given in Equation (A.12.2-3):

$$\frac{L_b}{r_{ltb}} \leq \frac{0,36 I_1 E}{Z_p F_{ymin}} \sqrt{\frac{A J}{(I_1 - I_2)(I_1 - J/2,6)}} \quad (\text{A.12.2-3})$$

where

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

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

L_b is the effective length of a beam-column between supports, i.e. the length between points that are either braced against lateral displacement of the compression flange, or braced against twist of the cross-section, in addition to lateral support;

A is the gross cross-sectional area;

J is the torsion constant, $J = \frac{4A_o^2}{\sum (b_w / t)}$

where

A_o is the area enclosed by the median line of the perimeter material of the section,

b_w is the width of each component (wall of the section) forming the closed perimeter,

t is the thickness of each component (wall of the section) forming the closed perimeter;

r_{ltb} is the radius of gyration about the minor axis as defined in Equation (A.12.3-6);

$F_{y,ltb}$ is the yield strength, F_y of the material that first yields when bending about the minor axis. Conservatively, F_y in Equations (A.12.2-2) and (A.12.2-3) may be taken as the maximum yield strength of all the components in a non-circular prismatic cross-section;

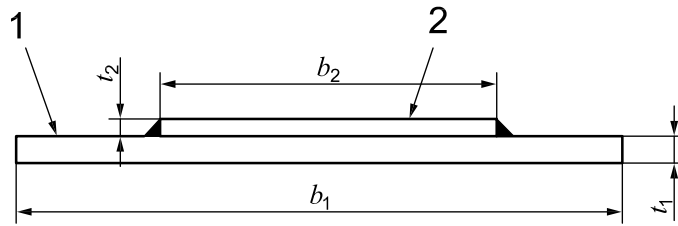
E is Young's modulus;

Z_p is the fully plastic effective section modulus about the major axis determined from Equation (A.12.3-2);

F_{ymin} is the minimum yield strength of the F_{yi} of all components in the cross-section of a non-circular prismatic member, in stress units, as defined in A.12.2.2.

A.12.2.3.3 Reinforced components

Reinforcement of member cross-sections is often of the form shown in Figure A.12.2-1.



Key

- 1 base plate
- 2 reinforcing plate
- b_1 width of base plate
- b_2 width of reinforcing plate
- t_1 thickness of base plate
- t_2 thickness of reinforcing plate

Figure A.12.2-1 — Definitions for reinforced plate

To be considered a reinforcing plate, the plate should nominally be in contact with the base plate across its full width and continuously welded to the base plate on all edges with adequate welds.

When a reinforcing component is used, there should be four independent checks of the cross-section classification in accordance with Tables A.12.2-2 to A.12.2-4:

- a) the reinforcing plate (using t_2) over the width b_2 , using buckling coefficient increased by a factor of 1,573 (see below in A.12.2.3.3);
- b) the combined plate using t_{check} over width b_1 ; see Equation (A.12.2-4);
- c) the base plate (using t_1) over the width b_2 using buckling coefficient increased by a factor of 1,573 (see A.12.2.3.3);
- d) the base plate (using t_1) over the dimension of the unreinforced widths (conservatively taken as $b_1 - b_2$).

If 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.12.2-2.

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 a) and c) to account for this limited buckling capability.

NOTE As an example, the first limit in Table A.12.3-1, $0,72t_f\sqrt{E/F_y}$, can be increased to $1,13 t_f\sqrt{E/F_y}$ as derived from $1,13 = 0,72 \times 1,573$.

The reinforcing plate should be classified as a compression flange internal component or web internal component in accordance with Tables A.12.2-2 and A.12.2-4 depending on the type of in-plane loading. The value of yield stress used is that of the reinforcing plate.

The composite section should be classified as a compression flange internal component, a web internal component or a compression flange outstand component in accordance with Tables A.12.2-2 to A.12.2-4 depending on the type of in-plane loading and support conditions. The value of thickness t_{check} for use with width b_1 in the equations in Table A.12.2-2 and A.12.2-4 should be determined from Equation (A.12.2-4):

$$t_{\text{check}} = (t_{\text{eff}}^3 t_1)^{1/4} \tag{A.12.2-4}$$

where

$$t_{\text{eff}} = (12 I / b_1)^{1/3} \tag{A.12.2-5}$$

$$I = [b_1(t_1 + t_2)^3 - (b_1 - b_2)t_2^3] / 3 - A(t_1 + t_2 - y_1)^2 \tag{A.12.2-6}$$

$$y_1 = [b_1 t_1^2 + b_2 t_2(2t_1 + t_2)] / (2A) \tag{A.12.2-7}$$

$$A = b_1 t_1 + b_2 t_2 \tag{A.12.2-8}$$

The value of yield stress for use in Tables A.12.2-2 to A.12.2-4 is the larger of the yield stress values for the reinforcing plate or the base plate.

Table A.12.2-2 — Cross-section classification — Flange internal components

Limiting width-to-thickness ratios for compressed internal components			
<p>A-A is the axis of bending</p>			
Class	Type	Section in bending	Section in compression
Plastic stress distribution in component and across section (compression positive)			
Plastic — Class 1	Rolled or welded	$bt_f \leq 1,03\sqrt{E/F_y}$	$bt_f \leq 1,03\sqrt{E/F_y}$
Compact — Class 2	Rolled or welded	$bt_f \leq 1,17\sqrt{E/F_y}$	$bt_f \leq 1,17\sqrt{E/F_y}$
Elastic stress distribution in component and across section (compression positive)			
Semi-Compact — Class 3	Rolled or welded	$bt_f \leq 1,44\sqrt{E/F_y}$	$bt_f \leq 1,44\sqrt{E/F_y}$
Slender — Class 4	Rolled or welded	$bt_f > 1,44\sqrt{E/F_y}$	$bt_f > 1,44\sqrt{E/F_y}$

Table A.12.2-3 — Cross-section classification — Outstand components

Limiting width-to-thickness ratios for outstand components				
<p>A-A is the axis of bending</p>				
Class	Type	Outstand subject to compression	Outstand subject to compression and bending	
			Tip in compression	Tip in tension
Plastic stress distribution in component (compression positive)				
Plastic — Class 1	Rolled Welded	$bt_f \leq 0,33\sqrt{E/F_y}$ $bt_f \leq 0,30\sqrt{E/F_y}$	$bt_f \leq (0,33/\alpha)\sqrt{E/F_y}$ $bt_f \leq (0,30/\alpha)\sqrt{E/F_y}$	$bt_f \leq [0,33/(\alpha\sqrt{\alpha})]\sqrt{E/F_y}$ $bt_f \leq [0,30/(\alpha\sqrt{\alpha})]\sqrt{E/F_y}$
Compact — Class 2	Rolled Welded	$bt_f \leq 0,37\sqrt{E/F_y}$ $bt_f \leq 0,33\sqrt{E/F_y}$	$bt_f \leq (0,37/\alpha)\sqrt{E/F_y}$ $bt_f \leq (0,33/\alpha)\sqrt{E/F_y}$	$bt_f \leq [0,37/(\alpha\sqrt{\alpha})]\sqrt{E/F_y}$ $bt_f \leq [0,33/(\alpha\sqrt{\alpha})]\sqrt{E/F_y}$
Elastic stress distribution in component (compression positive)			Maximum compression at tip 	Maximum compression at connected edge
Semi-Compact — Class 3	Rolled Welded	$bt_f \leq 0,55\sqrt{E/F_y}$ $bt_f \leq 0,50\sqrt{E/F_y}$	$bt_f \leq 0,84\sqrt{k_\sigma E/F_y}$ $bt_f \leq 0,76\sqrt{k_\sigma E/F_y}$ $\psi = \sigma_2/\sigma_1$ $k_\sigma = 0,57 - 0,21\psi + 0,07\psi^2$ for $1 \geq \psi \geq -1$	$bt_f \leq 0,84\sqrt{k_\sigma E/F_y}$ $bt_f \leq 0,76\sqrt{k_\sigma E/F_y}$ $\psi = \sigma_2/\sigma_1$ $k_\sigma = 0,578/(\psi + 0,34)$ for $1 \geq \psi \geq 0$ $k_\sigma = 1,7 - 5\psi + 17,1\psi^2$ for $0 > \psi \geq -1$
Slender — Class 4	Rolled or Welded	$bt_f >$ than for Class 3	$bt_f >$ than for Class 3	$bt_f >$ than for Class 3

In the figures relating to stress distributions, the dimension, *b*, is illustrated only in the case of rolled sections. For welded sections, *b* should be assigned as shown in the diagrams at the top of the table.

When determining α for Class 1 and 2 members, the loads should be scaled to give a fully plastic stress distribution. For all classes, it is conservative to use the relevant compression case.

Table A.12.2-4 — Cross-section classification — Web internal components

Limiting width-to-thickness ratios for web internal components			
<p>A-A is the axis of bending</p>			
Class	Web subject to bending	Web subject to compression	Web subject to bending and compression
Plastic stress distribution in component (compression positive)			
Plastic — Class 1	$\alpha = 0,5$ $d/t_w \leq 2,56\sqrt{(E/F_y)}$	$\alpha = 1,0$ $d/t_w \leq 1,03\sqrt{(E/F_y)}$	when $\alpha > 0,5$ $d/t_w \leq \frac{5,18\sqrt{(E/F_y)}}{(6,043\alpha - 1)}$ when $\alpha \leq 0,5$ $d/t_w \leq 1,28\sqrt{(E/F_y)}\alpha$
Compact — Class 2	$d/t_w \leq 3,09\sqrt{(E/F_y)}$	$d/t_w \leq 1,17\sqrt{(E/F_y)}$	when $\alpha > 0,5$ $d/t_w \leq \frac{4,82\sqrt{(E/F_y)}}{(5,12\alpha - 1)}$ when $\alpha \leq 0,5$ $d/t_w \leq \frac{1,55\sqrt{(E/F_y)}}{\alpha}$
Elastic stress distribution in component (compression positive)			
Semi-Compact — Class 3	$d/t_w \leq 4,14\sqrt{(E/F_y)}$	$d/t_w \leq 1,44\sqrt{(E/F_y)}$	when $\psi > -1,0$ $d/t_w \leq \frac{1,44\sqrt{(E/F_y)}}{(6,674 + 0,327\psi)}$ when $\psi \leq -1,0$ $d/t_w \leq 2,07(1 - \psi)\sqrt{(-\psi)\sqrt{(E/F_y)}}$
Slender — Class 4	$d/t_w >$ than for Class 3	$d/t_w >$ than for Class 3	$d/t_w >$ than for Class 3
When determining α for Class 1 and 2 members, the loads should be scaled to give a fully plastic stress distribution. For all classes it is conservative to use the relevant compression case.			

A.12.3 Section properties of non-circular prismatic members

A.12.3.1 General

Cross-sectional properties appropriate for the strength assessment of non-circular prismatic members of all classes should be determined as described in A.12.3.2 to A.12.3.4; the nomenclature and definition of variables is summarized in A.12.3.5. The properties appropriate for the stiffness assessment of prismatic members should be based on elastic considerations.

Where elastic section properties are determined for class 1 and class 2 sections in place of plastic section properties (e.g. for Euler amplification calculations or structural analysis input of stiffness parameters), these should be determined in accordance with A.12.3.3.

Cross-sectional properties are normally required in respect of both major and minor axes of a non-circular prismatic member.

Cross-sectional properties for tubular members are specified in A.12.5.

The cross-sectional properties used in the stiffness model (e.g. when determining structural deflections and natural periods) can differ from those used when assessing member strengths. For example, leg chord properties may include approximately 10 % of the maximum rack tooth area when determining the leg stiffness. This additional material should not be included when calculating the section properties for strength assessment, except it may be used when determining the column buckling strength (A.12.6.2.4) and moment amplification (A.12.4).

A.12.3.2 Plastic and compact sections

A.12.3.2.1 Axial properties — Class 1 and class 2 sections

For class 1 plastic and class 2 compact sections, section properties should be determined assuming that fully plastic behaviour can occur. The properties required for a strength assessment should be determined taking into account the physical distribution of components comprising the cross-section and their yield strengths. For simplicity, the following approximations can be used to determine the relevant properties.

For axial tension and compression, the fully plastic effective cross-sectional area for use in a strength assessment, A_p is as given in Equation (A.12.3-1):

$$A_p = (\sum F_{yi} A_i) / F_{ymin} \quad (\text{A.12.3-1})$$

where

F_{yi} is the yield strength of the i th component of the cross-section of a prismatic member, as defined in A.12.2.2;

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

F_{ymin} is the minimum yield strength of the F_{yi} of all components in the cross-section of a prismatic member, in stress units as defined in A.12.2.2.

NOTE 1 The centroid of the plastic section (or squash centre) of a member comprising components of differing yield strength can be offset from the centroid of the elastic section.

NOTE 2 A_p can be larger than the physical cross-section of the member.

A.12.3.2.2 Flexural properties — Class 1 and class 2 sections

The second moment of area, I_f , should be determined using the fully effective cross-section.

The fully plastic effective section modulus Z_p is as given in Equation (A.12.3-2):

$$Z_p = (\sum F_{yi} d_i A_i) / F_{ymin} \quad (\text{A.12.3-2})$$

where d_i is the distance between the centroid of the 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 that should be used in the calculation of plastic moment strengths should be F_{ymin} as defined in A.12.2.2.

A.12.3.3 Semi-compact sections

For class 3 semi-compact sections the section properties should be based on elastic properties assuming that the full cross-section is effective. The relevant variables are the cross-sectional area, A_f , as given in Equation (A.12.3-3), the second moment of area, I_f , and the elastic section modulus, S_f .

$$A_f = \sum A_i \quad (\text{A.12.3-3})$$

The properties I_f and S_f should be determined assuming that the full cross-section is effective for bending about both major and minor axes. When considering a cross-section comprised of components having different yield strengths, the section moduli used in the calculations should encompass all critical points on the cross-section.

NOTE Critical stress locations are typically those at the edges of components and are a function of the member forces, the yield strength of the component and its position within the cross-section of the member.

A.12.3.4 Slender sections

A.12.3.4.1 General

Class 4 classification is determined from Tables A.12.2-1 to A.12.2-4. Cross-sectional properties for class 4 slender sections should be determined using elastic principles. In tension, fully effective sections should be assumed, i.e. A_f and S_f . In compression, the sectional properties should be based on effective sections as described here.

When analysing structures that contain class 4 sections, care should be taken when determining the force distributions. It is recommended that the structural analysis be performed using full elastic section properties and that the reduced section properties are used only for the member strength checks. Since this overestimates the forces in class 4 members, care should be taken when the use of the reduced sections causes a significantly different force distribution. In this case, an iterative analysis process can be required.

Effective sections should be based on actual plating thicknesses combined with plating effective widths. The effective widths of compression flange internal or outstand components should be determined in accordance with the equations presented in Table A.12.3-1 a) or b), respectively. The effective widths of web internal components subject to compression and/or bending should be determined as shown in Table A.12.3-1 c) for which the following definitions apply (compression is taken as positive and tension as negative):

- ψ is the ratio of compressive stress to bending stress;
- σ_1 is the compressive stress if σ_2 is tensile or the larger compressive stress if σ_2 is also compressive;
- σ_2 is the tensile stress if σ_2 is tensile or the smaller compressive stress if σ_2 is compressive;
- k is the buckling coefficient;

- ρ is the reduction coefficient;
- λ_p is the plate slenderness parameter;
- λ_{plim} is the limiting plate slenderness ratio;
- λ_{po} is the plate slenderness ratio coefficient.

When determining effective widths for web internal components, the stress ratio, ψ , used in Table A.12.3-1 should be based on compression flange internal and outstand component effective widths, but the gross web section properties may be used.

The area reduction of curved components should be determined through the use of A.12.5.2.3. The following steps should be followed.

- a) The representative local buckling strength should be determined for a tubular member with the wall thickness and diameter equivalent to the curved component in the non-circular prismatic member.
- b) The strength of a tubular, of the same diameter and wall thickness used in step a), should be determined based on its full cross section and material yield.
- c) The ratio of the strengths should be determined as in step a) strength divided by step b) strength.
- d) This ratio of strengths should then be used to determine an equivalent reduced area of the curved component in the non-circular prismatic member.

The use of plating effective widths generally leads to a shift in the neutral axis compared with that found using gross sectional properties. This shift should be taken into account when determining effective widths. When the structural analysis is performed using gross section properties, the additional moment caused by the shift in the neutral axis should be found as the product of the axial force acting on the member and the shift in the neutral axis. This moment should be treated as additional to other moments acting on the effective section unless more onerous conditions arise if it is omitted.

A.12.3.4.2 Effective areas for compressive loading

The effective area $A_{eff,i}$ of a compressed component should be found as the product of its thickness and its effective width (which should never be taken as greater than the actual width). The effective area of a curved component subject to uniform compression should be determined from its actual area reduced by the ratio of its strength when treated as a class 3 or class 4 tubular [Equation (12.5-5), when $0,170 < A F_y/P_{xe}$] versus its strength when treated as class 1 or class 2 tubular [Equation (12.5-5), when $A F_y/P_{xe} \leq 0,170$], as set out in steps a) to d) in A.12.3.4.1. The total effective area, A_{ec} , is the sum of the component effective areas, as given in Equation (A.12.3-4):

$$A_{ec} = \sum A_{eff,i} \tag{A.12.3-4}$$

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 (which should never be taken as greater than the actual widths) should be determined from Table A.12.3-1 b). The effective area of a curved component subject to flexure should be determined from its actual area reduced by the ratio of its strength when treated as class 3 or class 4 tubular [Equation (12.5-10), for $0,1034 < (F_y D)/(E t)$] versus its strength when treated as class 1 tubular [Equation (12.5-10), for $(F_y D)/(E t) \leq 0,0517$], as set out in steps a) to d) in A.12.3.4.1. The effective second moment of area I_e should be found by calculating the properties of the section based on fully effective areas for components subject to tension, on effective areas as defined in A.12.3.4.2 for components subject to compression, and on effective areas as defined in the first paragraph for components subject to combinations of compression and flexure.

Table A.12.3-1 — Section properties — Effective widths for components in slender sections

<p>a) Compression internal components</p>		
<p>b) Outstand components under compression and/or bending</p>		
	$1 \geq \psi \geq 0$ $d_{\text{eff}} = \rho d$ $d_{e1} = 2d_{\text{eff}}/(5 - \psi)$ $d_{e2} = d_{\text{eff}} - d_{e1}$	$\psi = \sigma_2/\sigma_1$ $\rho = 1 \quad \text{if } \lambda_p \leq 0,75$ $\rho = [\lambda_p - 0,047(3 + \psi)]/\lambda_p^2 \quad \text{if } \lambda_p > 0,75$ <p style="text-align: right;">and $(3 + \psi) \geq 0$</p>
	$\psi \leq 0$ $d_{\text{eff}} = \rho d_c = \rho d/(1 - \psi)$ $d_{e1} = 0,4 d_{\text{eff}}$ $d_{e2} = 0,6 d_{\text{eff}}$	$\lambda_p = 1,04(d/t_w) \sqrt{[F_y]/(Ek)}$ $k = 8,2/(1,05 + \psi) \quad \text{if } 1 \geq \psi > 0$ $k = 7,81 - 6,29 \psi + 9,78 \psi^2 \quad \text{if } 0 \geq \psi > -1$ $k = 5,98 (1 - \psi)^2 \quad \text{if } -1 \geq \psi > -3$
<p>c) Internal components under compression and/or bending</p>		
<p>Key</p> <p>A-A axis of bending</p> <p> ineffective area, which is ignored when calculating effective section properties</p>		
<p>NOTE a) is a special case of c) with no bending and is included for clarity.</p>		

Application of this procedure to determine effective second moments of area when applied to cross-sections with slender components, especially when the section is not symmetric with respect to a particular axis, leads to two values of I_e about such an axis, depending upon the sign of the bending moment. Conservatively, the smaller value of I_e can be used throughout the strength analysis.

When considering a cross-section comprised of components having different yield strengths, the reduced elastic section modulus S_e used in the calculations should encompass all critical points on the cross-section:

$$S_e = I_e / y_i \quad (\text{A.12.3-5})$$

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

NOTE Critical stress locations are typically those at the edges of components and are a function of the member forces, the yield strength of the component and its position within the cross-section of the member.

A.12.3.5 Cross-sectional properties for the assessment

A.12.3.5.1 Tension

In tension, the cross-sectional area for use in the assessment should be A_t where

- $A_t = A_p$ for class 1 plastic or class 2 compact sections, see Equation (A.12.3-1),
- $= A_f$ for class 3 semi-compact sections as defined in Equation (A.12.3-3),
- $= A_f$ 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} 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 for use in the 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),
- $= A_f$ for class 3 semi-compact sections as defined in Equation (A.12.3-3),
- $= A_{ec}$ 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 the y and z axes of bending that should be used in the assessment should be determined from the following:

- $I_y, I_z = I_f$ for class 1 plastic and class 2 compact sections as defined in A.12.3.2.2,
- $= I_f$ for class 3 semi-compact sections as defined in A.12.3.3,
- $= I_e$ for class 4 slender sections as described in A.12.3.4.3 accounting for both the chosen axis and the direction of bending.

The section moduli for the two bending axes should be determined from the following:

$$\begin{aligned}
 S_y, S_z &= Z_p \text{ for class 1 plastic or class 2 compact sections, see Equation (A.12.3-2),} \\
 &= S_f \text{ for class 3 semi-compact sections as defined in A.12.3.3 for each critical stress location,} \\
 &= S_e \text{ for class 4 slender sections as defined in A.12.3.4.3 for each critical stress location,} \\
 &\quad \text{accounting for both the chosen axis and the direction of bending.}
 \end{aligned}$$

The radius of gyration about the minor axis that should be used for lateral-torsional buckling considerations, r_{ltb} , should be determined as given in Equations (A.12.3-6):

$$\begin{aligned}
 r_{ltb} &= (I_p/A_c)^{0.5} \text{ for sections in classes 1 to 3,} \\
 &= (I_e/A_{ec})^{0.5} \text{ for sections in class 4.}
 \end{aligned}
 \tag{A.12.3-6}$$

A.12.4 Effects of axial force on bending moment

A.12.4.1 General

Euler moment amplification (p - δ) applies to all members in axial compression.

For classes 1, 2, and 3 cross-sections, the eccentricity between the elastic and plastic centroids induces an additional moment. This affects members in both tension and compression.

For class 4 members, in addition to the Euler moment amplification, there is an eccentricity between the full cross-section area normally used in the structural analysis and the effective neutral axis used in the member strength check. This can affect members in both tension and compression.

A.12.4.2 Member moment correction due to eccentricity of axial force

The plastic centroid or “centre of squash” is defined as the location at which the axial force produces no moment on the fully plastic section. For chords with material asymmetry (e.g. when the section includes components of differing yield strengths) the centre of squash can be offset from the elastic centroid. Before a section is checked, the moments should be corrected by the moment due to the axial force times the eccentricity between the elastic centroid (used in the structural analysis) and the “centre of squash” in accordance with Equation (A.12.4-1). There is no eccentricity for tubular members or for non-circular prismatic members with material symmetry.

The corrected effective moment, M_{ue} , should be calculated for each axis of bending, as given in Equation (A.12.4-1):

$$M_{ue} = M_u + eP_u \tag{A.12.4-1}$$

where

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

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

e is the eccentricity between the axis used for structural analysis and that used for structural strength checks, taking due account of the sign in combination with the sign convention for P_u ;

- e for class 1 and 2 members, is the distance between the elastic and plastic neutral axes orthogonal to the axis of bending under consideration. Annex F presents data including this offset distance (together with other geometric data) for many members of each chord family,
- e for class 3 members, is equal to e_a as defined in A.12.6.2.3,
- e for class 4 members, is the distance between the neutral axes of the full and effective cross-sections, orthogonal to the axis of bending under consideration,
- e is equal to 0 if the structural model fully accounts for the offset between the neutral axes of the modelled member in the strength checks,
- e is equal to 0 for tubular members; for other cross-sections in classes 1, 2 and 3 with material symmetry and when an elastic strength check is used for the assessment of members in classes 1, 2 and 3.

A.12.4.3 Member moment amplification and effective lengths

The amplified moment, M_{ua} , should be calculated for each axis of bending as given in Equation (A.12.4-2):

$$M_{ua} = B M_{ue} \tag{A.12.4-2}$$

where

M_{ue} is as defined in A.12.4.2;

B is the member moment amplification factor for the axis under consideration, equal to one of the following:

- $B = 1,0$ for (i) members in tension, or (ii) members in compression where the individual member forces are determined from a second order analysis, i.e. the equilibrium conditions are formulated on the elastically deformed structure so that local $p-\delta$ 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 $p-\delta$ effects:

where

$P_E = (\pi^2 E I) / (K L)^2$, and should be calculated for the plane of bending;

I is the second moment of area for the plane of bending as defined in A.12.3.5.3;

K and C_m are given in Table A.12.4-1;

K is the effective length factor for the plane of flexural buckling;

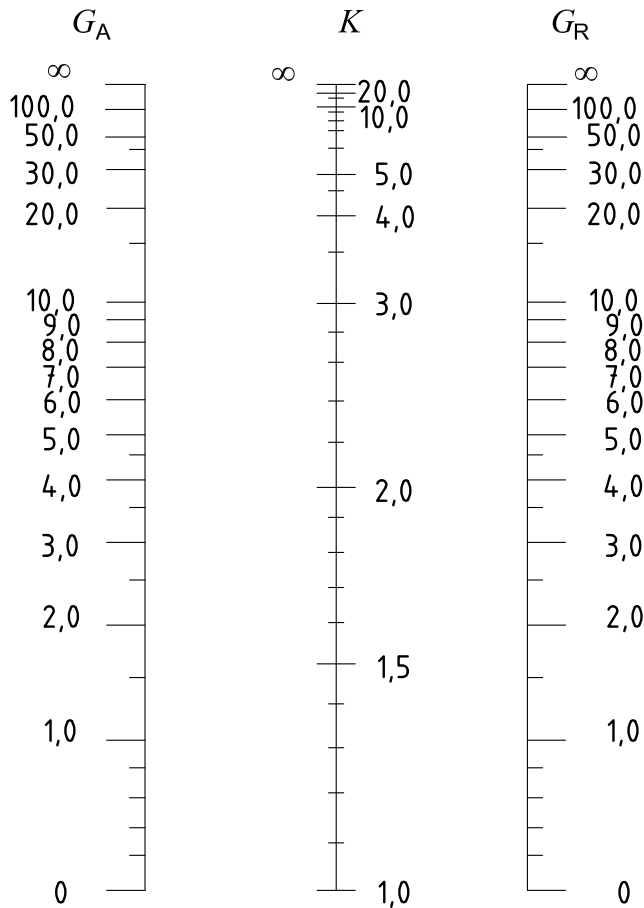
L is the unbraced length of member for the plane of flexural buckling normally taken as one of the following:

- face to face length for braces,
- braced point to braced point length for chords,
- longer segment length of X-braces (one pair is in tension, if not braced out-of-plane).

When the analysis of a jack-up with single-column tubular or box section legs has been undertaken accounting for the member moment amplification effects of global P-Δ/hull-sway, *B* may be taken as 1,0 as local p-δ and global P-Δ are the same. For these jack-ups, local strength due to guide reactions should be assessed in conjunction with the member forces.

Table A.12.4-1 — Effective length and moment reduction factors

Structural member		<i>K</i>	<i>C_m</i> ^a	
Tubular or box complete legs		2,0 ^b	A	
Chords with lateral loading		1,0	C	
Chords without lateral loading		1,0	B	
Tubular braces	Primary diagonals and horizontals	0,7	B or C	
	K-braces ^c	0,7	C	
	X-brace ^c	Longer segment length	0,8	C
		Full length ^d	0,7	C
	Secondary horizontals	0,7	C	
<p>^a The value of <i>C_m</i> can be determined from rational analysis. In lieu of such analysis, the following values may be used:</p> <p>A For members whose ends are restrained against sidesway $C_m = 0,85$ For members whose ends are unrestrained against sidesway $C_m = 1,0$</p> <p>B For members with no significant transverse loading, ignoring self-weight; buoyancy; and direct wave/current and wind actions: $C_m = 0,6 - 0,4 M_1/M_2$ 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</p> <p>C For members with significant transverse loading, other than self-weight; buoyancy; and direct wave/current and wind actions: $C_m = 1,0 - 0,2 P_u/P_E$ (AISC[A.12.5-1] Table C-C2.1) $P_E = P_{Ey}$ or P_{Ez} as appropriate for the axis of bending under consideration.</p> <p>^b Alternatively use effective length alignment chart in Figure A.12.4-1.</p> <p>^c For either in-plane or out-of-plane effective lengths, at least one pair of members framing into a K- or X-joint is in tension if the joint is not braced out-of-plane.</p> <p>^d For X-braces, when all members are in compression and the joint is not braced out-of-plane.</p>				



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

$$G = \frac{\frac{I_c}{L_c}}{\sum \frac{I_0}{L_0}}$$

in which Σ indicates a summation of all members rigidly connected to that joint and lying in the plane in which buckling of the column is being considered. I_c is the moment of inertia and L_c the unsupported length of the column section, and I_0 is the moment of inertia and L_0 the unsupported length of a girder or other restraining member. I_c and I_0 are taken about axes perpendicular to the plane of buckling being considered.

For column ends supported by a pinned restraint, G is theoretically infinite but, unless truly friction free, can be taken as 10 for practical cases. If the column end is rigidly restrained, G may be taken as 1,0. Smaller values may be used if justified by analysis.

NOTE Taken from ISO 19902:2007, Figure A.13.5-4.

Figure A.12.4-1 — Alignment chart for determining the effective length of unbraced columns

A.12.5 Strength of tubular members

A.12.5.1 Applicability

The strength of unstiffened tubular members that satisfy Equation (A.12.5-1) should be assessed in accordance with A.12.5.

$$Dt < 120 \tag{A.12.5-1}$$

Tubulars that do not satisfy Equation (A.12.5-1) should be assessed using alternative methods that result in levels of reliability comparable to those implicit in this part of ISO 19905, such as References [A.12.5-3] and [A.12.5-4].

The strength equations in A.12.5.1 to A.12.5.3 for $D/t < 120$ are unconservative for tubulars with reductions in their cross-section. Where a tubular includes cross-sections with cut-outs, pin-holes, etc., it should be treated as for a non-circular prismatic member, unless it is adequately reinforced. Reinforcement can comprise either doubler plates that surround the hole or stiffeners that extend at least half the width of the hole above and below the hole. If the reinforcement replaces all the lost area the tubular, the strength equations in A.12.5 may be used.

The strength equations are considered applicable for steels with a yield strength up to 700 N/mm². The yield strength used should be as specified in A.12.2.2.

NOTE The strength equations for tubular members are based on ISO 19902:2007, Clause 13. However, for use in this part of ISO 19905, the ISO 19902 formulations have been converted to a force base rather than a stress base.

The equations ignore the effect of hydrostatic pressure. The condition under which hydrostatic pressure can be ignored for a specific member is as given in Equation (A.12.5-2):

$$(D/t)_m = 211/d_w^{0,335} \quad (\text{A.12.5-2})$$

where

- d_w is the effective head of water in metres applicable to the tubular in question; it is the depth below the water surface (including penetration into the seabed where applicable) plus $p\gamma'/(\rho_w g)$;
- p is the depth below the sea floor in metres (zero if above sea floor);
- γ' is the submerged (effective) unit weight of the soil;
- ρ_w is the mass density of water;
- g is the acceleration due to gravity;
- $(D/t)_m$ is the maximum D/t ratio possible given d_w .

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 effective head of water

Effective head of water d_w m	Maximum tubular $(D/t)_m$
43	60,0
50	56,9
75	49,7
100	45,1
125	41,9
150	39,4
200	35,8

If the member D/t exceeds the limiting value $(D/t)_m$, the assessor should refer to ISO 19902, which is based on stress rather than strength.

A.12.5.2 Tension, compression and bending strength of tubular members

A.12.5.2.1 Axial tensile strength check

Tubular members subjected to axial tensile forces, P_{ut} , due to factored actions should satisfy Equation (A.12.5-3):

$$P_{ut} \leq A F_y / \gamma_{R,Tt} \quad (\text{A.12.5-3})$$

where

F_y is the yield strength in stress units as defined in A.12.2.2;

A is the gross cross-sectional area;

$\gamma_{R,Tt}$ is the partial resistance factor for axial tensile strength, $\gamma_{R,Tt} = 1,05$.

A.12.5.2.2 Axial compressive strength check

Tubular members subjected to axial compressive forces, P_{uc} , due to factored actions should satisfy Equation (A.12.5-4):

$$P_{uc} \leq P_a / \gamma_{R,Tc} \quad (12.5-4)$$

where

P_a is the representative axial compressive strength as determined in A.12.5.2.4;

$\gamma_{R,Tc}$ is the partial resistance factor for axial compressive strength, $\gamma_{R,Tc} = 1,15$.

A.12.5.2.3 Local buckling strength

The representative local buckling strength, P_{yc} , should be determined as given in Equations (A.12.5-5):

$$P_{yc} = A F_y \quad (\text{for } A F_y / P_{xe} \leq 0,170)$$

$$P_{yc} = [1,047 - 0,274 A F_y / P_{xe}] A F_y \quad (\text{for } 0,170 < A F_y / P_{xe} \leq 200 F_y / E) \quad (A.12.5-5)$$

where, in addition to the variables in A.12.5.2.1, P_{xe} is the representative elastic local buckling strength, calculated as given in Equation (A.12.5-6):

$$P_{xe} = 2 C_x E A (t/D) \quad (A.12.5-6)$$

where C_x is the critical elastic buckling coefficient.

The theoretical value of C_x for an ideal tubular is 0,6. However, a reduced value of $C_x = 0,3$ is recommended for use in the determination of P_{xe} to account for the effect of initial geometric imperfections. A reduced value of $C_x = 0,3$ is also implicit in the limits for $A F_y / P_{xe}$ given in Equation (A.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 Equations (A.12.5-7) and (A.12.5-8):

$$P_a = (1,0 - 0,278 \lambda^2) P_{yc} \quad (\text{for } \lambda \leq 1,34)$$

$$P_a = 0,9 P_{yc} / \lambda^2 \quad (\text{for } \lambda > 1,34) \quad (A.12.5-7)$$

where

$$\lambda = (P_{yc} / P_E)^{0,5} \quad (A.12.5-8)$$

P_{yc} is the representative local buckling strength (see A.12.5.2.3);

λ is the column slenderness parameter;

P_E is the smaller of the Euler buckling strengths about the y- or z-direction, $P_E = \pi^2 E I / (KL)^2$;

E is Young's modulus as defined in A.12.1.1;

K is the effective length factor in y- or z-direction; see A.12.4.3;

L is the unbraced length in y- or z-direction; see A.12.4.3;

I is the second moment of area of the tubular.

A.12.5.2.5 Bending strength check

Tubular members subjected to bending moments, M_u , should satisfy Equation (A.12.5-9):

$$M_u \leq M_b / \gamma_{R,Tb} \quad (\text{A.12.5-9})$$

where

M_u is equal to M_{uy} or M_{uz} ; it is the bending moment due to factored actions about member y- and z-axes, respectively, determined in an analysis that includes global P- Δ effects;

M_b is the representative bending moment strength, determined as given in Equations (A.12.5-10):

$$\begin{aligned} M_b &= M_p && \text{for } (F_y D) / (E t) \leq 0,051\ 7 \\ M_b &= [1,13 - 2,58 (F_y D) / (E t)] M_p && \text{for } 0,051\ 7 < (F_y D) / (E t) \leq 0,103\ 4 \\ M_b &= [0,94 - 0,76 (F_y D) / (E t)] M_p && \text{for } 0,103\ 4 < (F_y D) / (E t) \leq 120 (F_y / E) \end{aligned} \quad (\text{A.12.5-10})$$

M_p is the plastic moment strength as given in Equation (A.12.5-11):

$$M_p = F_y [D^3 - (D - 2t)^3] / 6 \quad (\text{A.12.5-11})$$

$\gamma_{R,Tb}$ is the partial resistance factor for bending strength, $\gamma_{R,Tb} = 1,05$.

A.12.5.3 Tubular member combined strength checks

A.12.5.3.1 Axial tension and bending strength check

Tubular members subjected to combined axial tension and bending should satisfy the condition given in Equation (A.12.5-12) at all cross-sections along their length:

$$\gamma_{R,Tt} P_{ut} / (A F_y) + \gamma_{R,Tb} (M_{uy}^2 + M_{uz}^2)^{0,5} / M_b \leq 1,0 \quad (\text{A.12.5-12})$$

where, in addition to the previously defined variables

P_{ut} is the axial tensile force due to factored actions;

A is the gross cross-sectional area;

F_y is the yield strength in stress units as defined in A.12.2.2;

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

M_b is the representative bending moment strength, as defined in Equation (A.12.5-10);

$\gamma_{R,Tt}$ is the partial resistance factor for axial tensile strength, $\gamma_{R,Tt} = 1,05$;

$\gamma_{R,Tb}$ is the partial resistance factor for bending strength, $\gamma_{R,Tb} = 1,05$.

A.12.5.3.2 Axial compression and bending strength check

Tubular members subjected to combined axial compression and bending should satisfy the conditions given in Equations (A.12.5-7) and (A.12.5-7) at all cross-sections along their length:

beam-column check:

$$(\gamma_{R,Tc} P_{uc}/P_a) + (\gamma_{R,Tb}/M_b) (M_{uay}^2 + M_{uaz}^2)^{0,5} \leq 1,0 \quad (\text{A.12.5-13})$$

and local strength check:

$$(\gamma_{R,Tc} P_{uc}/P_{yc}) + (\gamma_{R,Tb}/M_b) (M_{uey}^2 + M_{uez}^2)^{0,5} \leq 1,0 \quad (\text{A.12.5-14})$$

where

P_{uc} is the axial compressive force due to factored actions;

P_{yc} is the representative local buckling strength in A.12.5.2.3;

P_a is the representative axial compressive strength as determined in A.12.5.2.4;

M_{uey} is the corrected effective bending moment about member y-axis due to factored actions as determined in A.12.4.2;

M_{uez} is the corrected effective bending moment about the member z-axis due to factored actions as determined in A.12.4.2;

M_{uay} is the amplified bending moment about the member y-axis due to factored actions as determined in A.12.4.3;

M_{uaz} is the amplified bending moment about the member z-axis due to factored actions as determined in A.12.4.3;

M_b is the representative bending moment strength, as defined in Equation (A.12.5-10);

$\gamma_{R,Tb}$ is the partial resistance factor for bending strength, $\gamma_{R,Tb} = 1,05$;

$\gamma_{R,Tc}$ is the partial resistance factor for axial compressive strength, $\gamma_{R,Tc} = 1,15$.

A.12.5.3.3 Beam shear strength check

Tubular members subjected to beam shear forces due to factored actions should satisfy Equation (A.12.5-15):

$$V \leq P_v/\gamma_{R,Tv} \quad (\text{A.12.5-15})$$

where

V is the beam shear due to factored actions;

P_v is the representative shear strength, as given in Equation (A.12.5-16):

$$P_v = A F_y/(2\sqrt{3}) \quad (\text{A.12.5-16})$$

A is the gross cross-sectional area;

$\gamma_{R,Tv}$ is the partial resistance factor for torsional and beam shear strengths, $\gamma_{R,Tv} = 1,05$.

A.12.5.3.4 Torsional shear strength check

Tubular members subjected to torsional shear forces due to factored actions should satisfy Equation (A.12.5-17):

$$T_u \leq T_v / \gamma_{R,Tv} \quad (\text{A.12.5-17})$$

where

T_u is the torsional moment due to factored actions;

T_v is the representative torsional strength, calculated as given in Equation (A.12.5-18):

$$T_v = 2I_p F_y / (D \sqrt{3}) \quad (\text{A.12.5-18})$$

I_p is the polar moment of inertia, calculated as given in Equation (A.12.5-19):

$$I_p = (\pi / 32) [D^4 - (D - 2t)^4] \quad (\text{A.12.5-19})$$

A.12.6 Strength of non-circular prismatic members

A.12.6.1 General

The structural strength provisions for rolled and welded non-circular prismatic members are generally based on the AISC 2005^[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 strength equations 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^[A.12.6-1]. Interpretation of the specifications was necessary to enable presentation of a straightforward method for the assessment of beam-columns with components of varying yield strength and/or with cross-sections having only a single axis of symmetry. Development of the specifications was necessary to provide the following:

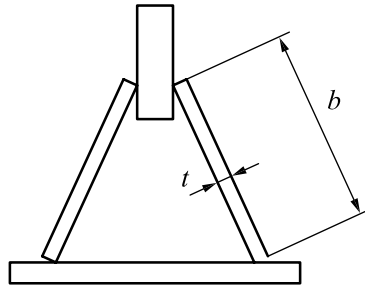
- a) a method to deal with member cross-sections comprising components constructed of steels with different yield strengths;
- b) a method for the assessment of beam-columns under biaxial bending to overcome a conservatism that has been identified in the standard AISC interaction equations.

The yield strength used in A.12.6 should be as specified in A.12.2.2.

The effects of hydrostatic loading on non-circular prismatic members should be considered. The critical condition for hydrostatic loading on non-circular prismatic chord members is likely to occur when high spudcan fixity results in high chord axial loads in deep water.

Hydrostatic pressure effects on split tubular and similar members should be addressed as described in A.12.5.1. If the section fails to meet the un-reinforced tubular check, additional analysis can be used to determine the effects of the stiffening provided by the non-tubular components.

Hydrostatic pressure effects on flat plate components of members should be assessed as shown in Figure A.12.6-1 for values of β less than 2,0. If the component is used under conditions with an effective head of water greater than that given in Figure A.12.6-1, or if the calculated β is greater than 2,0, then rational analysis should be used to assess the effects of hydrostatic pressure on member utilization. For convenience, Table A.12.6-1 gives the limiting effective head of water for components of differing plate slendernesses.



$$d_w = 298\beta^4 - 2\,092\beta^3 + 5\,542\beta^2 - 6\,603\beta + 3\,025 \text{ for } \beta \leq 2,0$$

Key

- b* width of base plate
- t* thickness of base plate
- d_w* limiting effective head of water in metres for which additional analysis is not required; it is the depth below the water surface (including penetration into the seabed where applicable) + $p \gamma' / (\rho_w g)$
- β plate slenderness parameter $\beta = (b/t)(F_y/E)^{0,5}$
- p* is the depth below the sea floor in metres (zero if above sea floor)
- γ' is the submerged (effective) unit weight of the soil
- ρ_w is the mass density of water

Figure A.12.6-1 — Example chord showing plate dimensions for hydrostatic pressure screening check

Table A.12.6-1 — Maximum plate slenderness parameter β for given effective head of water

Effective head of water <i>d_w</i> m	Plate slenderness parameter β
170	1,0
120	1,1
85	1,2
48	1,4
32	1,6
24	1,8
20	2,0

In A.12.6.2 and A.12.6.3, *y* and *z* are used to define the axes of a non-circular prismatic member.

A.12.6.2 Non-circular prismatic members subjected to tension, compression, bending or shear

A.12.6.2.1 General

Non-circular prismatic members subjected to axial tension, axial compression, bending or shear should satisfy the applicable strength and stability checks specified in A.12.6.2.2 to A.12.6.2.7.

A.12.6.2.2 Axial tensile strength check

Non-circular prismatic members subjected to axial tensile forces, P_{ut} , due to factored actions should satisfy Equation (A.12.6-1):

$$P_{ut} \leq P_t / \gamma_{R,Pt} \tag{A.12.6-1}$$

where

P_t is the representative axial tensile strength of a non-circular prismatic member, calculated as given in Equation (A.12.6-2)

$$P_t = \Sigma(F_{yi}A_i) \quad (\text{A.12.6-2})$$

F_{yi} is the yield strength of the i th component of the cross-section of a prismatic member, in stress units, as defined in A.12.2.2;

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

$\gamma_{R,Pt}$ is the partial resistance factor for axial tensile strength, $\gamma_{R,Pt} = 1,05$.

A.12.6.2.3 Axial compressive local strength check

Non-circular prismatic members subjected to axial compressive forces, P_{uc} , due to factored actions should satisfy Equation (A.12.6-3):

$$P_{uc} \leq P_{pl}/\gamma_{R,Pcl} \quad (\text{A.12.6-3})$$

where, in addition to the definitions given in A.12.6.2.2

$\gamma_{R,Pcl}$ is the partial resistance factor for local axial compressive strength, $\gamma_{R,Pcl} = 1,1$;

P_{pl} is the representative local axial compressive strength of a non-circular prismatic member as given in Equations (A.12.6-4) to (A.12.6-6);

$$P_{pl} = \Sigma F_{yi}A_i \quad \text{for class 1 and class 2 members} \quad (\text{A.12.6-4})$$

$$P_{pl} = \Sigma F_{yi}A_i - (\Sigma F_{yi}A_i - F_{ymin}\Sigma A_i) \left(\frac{\lambda_h - \lambda_p}{\lambda_r - \lambda_p} \right)_h \quad \text{for class 3 members} \quad (\text{A.12.6-5})$$

$$P_{pl} = F_{ymin}A_c \quad \text{for class 4 members} \quad (\text{A.12.6-6})$$

F_{ymin} is the minimum yield stress of the F_{yi} of all components in the cross-section of a prismatic member, in stress units, as defined in A.12.2.2;

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

A_c is the cross-sectional area for use in the assessment of a non-circular prismatic member as defined in A.12.3.5.2;

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

λ_h = b/t or $2R/t$ as applicable for component h , with effective width b , or outside radius R ;

λ_p is as determined for component h from Tables A.12.2-2 to A.12.2-4 as given in Equations (A.12.6-7) to (A.12.6-10):

— for rectangular rolled or welded web or flange components supported along both edges:

$$\lambda_p = 1,17\sqrt{E/F_{yi}} \quad (\text{A.12.6-7})$$

- for rectangular rolled flange or web components supported along one edge:

$$\lambda_p = 0,37\sqrt{E/F_{yi}} \quad (\text{A.12.6-8})$$

- for rectangular welded flange or web components supported along one edge:

$$\lambda_p = 0,33\sqrt{E/F_{yi}} \quad (\text{A.12.6-9})$$

- for components derived from tubulars (with reference to Table A.12.2-1):

$$\lambda_p = 0,077 E/F_{yi} \quad (\text{A.12.6-10})$$

λ_r is determined for component h from Tables A.12.2-2 to A.12.2-4 as given in Equations (A.12.6-11) to A.12.6-14):

- for rectangular rolled or welded web or flange components supported along both edges:

$$\lambda_r = 1,44\sqrt{E/F_{yi}} \quad (\text{A.12.6-11})$$

- for rectangular rolled flange or web components supported along one edge:

$$\lambda_r = 0,55\sqrt{E/F_{yi}} \quad (\text{A.12.6-12})$$

- for rectangular welded flange or web components supported along one edge:

$$\lambda_r = 0,50\sqrt{E/F_{yi}} \quad (\text{A.12.6-13})$$

- for components derived from tubulars (with reference to AISC^[A.12.5-1] and Table A.12.2-1):

$$\lambda_r = 0,102 E/F_{yi} \quad (\text{A.12.6-14})$$

The eccentricity between the elastic and plastic neutral axes, e_a , for class 3 members (see A.12.4) can be calculated as given in Equation (A.12.6-15):

$$e_a = e \left(\frac{\lambda_r - \lambda_h}{\lambda_r - \lambda_p} \right)_h \quad (\text{A.12.6-15})$$

where e is as defined in A.12.4.2.

A.12.6.2.4 Axial compressive column buckling strength

There is no axial compressive column buckling strength check because it is inherent in the combined strength check for compression in A.12.6.3. However, the representative axial compressive strength of all member classifications subjected to flexural buckling should be determined as given Equations (A.12.6-16) to (A.12.6-19):

- a) for all grades of steel (conservative for high strength steel):

$$P_n = \left(0,658 \lambda_c^2 \right) P_{pl} \quad \text{for } \lambda_c \leq 1,5 \text{ [derived from AISC}^{\text{[A.12.5-1]}} \text{, Equation E3-2]} \quad (\text{A.12.6-16})$$

$$P_n = \left(0,877/\lambda_c^2\right)P_{pl} \quad \text{for } \lambda_c > 1,5 \text{ [derived from AISC}^{[A.12.5-1]}, \text{ Equation E3-3]} \quad (\text{A.12.6-17})$$

b) alternatively, for high-strength steels ($F_y > 450$ MPa), the following may be used (see F.1):

$$P_n = \left(0,7625\lambda_c^{3,22}\right)P_{pl} \quad \text{for } \lambda_c \leq 1,2 \quad (\text{A.12.6-18})$$

$$P_n = \left(0,8608/\lambda_c^{1,854}\right)P_{pl} \quad \text{for } \lambda_c > 1,2 \quad (\text{A.12.6-19})$$

where, in addition to the definitions in A.12.6.2.3,

$$\lambda_c = \left(\frac{P_{pl}}{P_E}\right)^{0,5} \quad (\text{derived from AISC}^{[A.12.5-1]}, \text{ Ch. E3; see also F.1}) \quad (\text{A.12.6-20})$$

P_E is the minimum Euler buckling load for any plane of bending, as defined in A.12.4.3 (including rack teeth of chords; see A.12.3.1).

When section contains un-reinforced cut-outs, the slenderness parameter, λ_c , should be based on the minimum section unless otherwise determined by analysis.

A.12.6.2.5 Bending moment strength

A.12.6.2.5.1 General

The classification of member cross-sections in A.12.2 is used to identify the potential for local buckling. The slender section properties determined in A.12.3.4 account for the local buckling of class 4 cross-sections.

The bending moment strength of typical closed section jack-up chord members used in truss legs is not normally limited by lateral torsional buckling. However, this should be checked as described in A.12.2.3.2.

A.12.6.2.5.2 Class 1 plastic and class 2 compact section bending moment strength

The representative bending moment strength, M_b , is given by the plastic bending moment of the entire section as given in Equation (A.12.6-21):

$$M_b = Z_p F_{ymin} \quad (\text{A.12.6-21})$$

where

M_b is the representative bending moment strength;

Z_p is the fully plastic effective section modulus, determined from Equation (A.12.3-2);

F_{ymin} is the minimum yield strength of all components in the cross-section of a prismatic member, in stress units, as defined in A.12.2.2.

NOTE Hybrid sections built up from components of different yield strengths are addressed by the methodology described in A.12.3.2.

A.12.6.2.5.3 Class 3 semi-compact section bending moment strength

The representative bending strength, M_b , is obtained by interpolating between the plastic bending moment and the limiting buckling moment as given in Equation (A.12.6-22):

$$M_b = M_p - (M_p - M_R) \left(\frac{\lambda_h - \lambda_p}{\lambda_r - \lambda_p} \right)_h \quad (\text{A.12.6-22})$$

where, in addition to the definitions in A.12.6.2.5.2

M_p is the plastic moment strength;

$M_p = Z_p F_{ymin}$ as calculated by Equation (A.12.6-21);

$$M_R = S_f F_y < M_p \quad (\text{A.12.6-23})$$

S_f is the elastic section modulus of a semi-compact section of a non-circular prismatic member for the plane of bending under consideration; see A.12.3.3;

h is the subscript referring to the component which produces the smallest value of M_b ;

$\lambda_h = b/t$ or $2R/t$ as applicable for component h ;

λ_p is as determined for component h from Tables A.12.2-2 to A.12.2-4, as given in Equations (A.12.6-24) to (A.12.6-29):

- for rectangular rolled or welded flange components supported along both edges when the bending results in to uniform compression:

$$\lambda_p = 1,17 \sqrt{(E / F_{yi})} \quad (\text{A.12.6-24})$$

- for rectangular rolled flange or web components supported along one edge and subject to combinations of compression and bending:

$$\lambda_p = 0,37 \sqrt{(E / F_{yi})} \quad (\text{A.12.6-25})$$

- for rectangular welded flange or web components supported along one edge and subject to combinations of compression and bending:

$$\lambda_p = 0,33 \sqrt{(E / F_{yi})} \quad (\text{A.12.6-26})$$

- for rectangular rolled or welded web components supported along both edges and subject to combinations of compression and bending:

$$\lambda_p = \left[4,82 \sqrt{(E / F_{yi})} \right] / (5,12\alpha - 1) \quad (\text{for } \alpha > 0,5) \quad (\text{A.12.6-27})$$

$$\lambda_p = \left[1,55 \sqrt{(E / F_{yi})} \right] / \alpha \quad (\text{for } \alpha \leq 0,5) \quad (\text{A.12.6-28})$$

where α 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.

- for components derived from circular tubes and subject to pure bending (see Table A.12.2-1):

$$\lambda_p = 0,103 E / F_{yi} \quad (\text{A.12.6-29})$$

When the location of the tubular component results in combined bending and compression the value of λ_p can conservatively be taken from Equation (A.12.6-10). Alternatively, the value of λ_p may be interpolated between the values for pure bending and pure compression.

λ_r is determined for component h from Tables A.12.2-2 to A.12.2-4, as given in Equations (A.12.6-30) to (A.12.6-35):

- for rectangular rolled or welded flange components supported along both edges when the bending results in uniform compression:

$$\lambda_r = 1,44\sqrt{(E/F_{yi})} \quad (\text{A.12.6-30})$$

- for rectangular rolled flange or web components supported along one edge and subject to combinations of compression and bending:

$$\lambda_r = 0,55\sqrt{(E/F_{yi})} \quad (\text{A.12.6-31})$$

- for rectangular welded flange or web components supported along one edge and subject to combinations of compression and bending:

$$\lambda_r = 0,50\sqrt{(E/F_{yi})} \quad (\text{A.12.6-32})$$

- for rectangular rolled or welded web components supported along both edges and subject to combinations of compression and bending:

$$\lambda_r = \left[1,44\sqrt{(E/F_{yi})} \right] / (0,674 + 0,327\psi) \quad (\text{for } \psi > -1,0) \quad (\text{A.12.6-33})$$

$$\lambda_r = [2,07(1 - \psi)\sqrt{-\psi}] \sqrt{(E/F_{yi})} \quad (\text{for } \psi \leq -1,0) \quad (\text{A.12.6-34})$$

where ψ is the stress ratio as shown in Table A.12.2-4.

- for components derived from circular tubes and subject to pure bending (see Table A.12.2-1):

$$\lambda_r = 0,22E/F_{yi} \quad (\text{A.12.6-35})$$

When the location of the tubular component results in combined bending and compression, the value of λ_r can conservatively be taken from Equation (A.12.6-14). Alternatively, the value of λ_r may be interpolated between the values for pure bending and pure compression.

A.12.6.2.5.4 Class 4 slender-section bending moment strength

The representative bending moment strength, M_b , of class 4 sections is given by the limiting flexural bending moment in Equation (A.12.6-36):

$$M_b = S_e F_y \quad (\text{A.12.6-36})$$

where S_e is the reduced elastic section modulus of a slender section of a non-circular prismatic member for the plane of bending under consideration, see A.12.3.4.3.

A.12.6.2.6 Bending moment strength affected by lateral torsional buckling

The reduced representative bending moment strength M_b due to LTB should be calculated for all members that do not meet the screening checks of either Equation (A.12.2-2) or Equation (A.12.2-3) for open and closed sections, respectively, regardless of the class of section. When the representative bending moment strength is reduced due to LTB compared to the strength calculated in A.12.6.2.5, the reduced bending moment strength should be used in the strength checks.

Further guidance on the bending moment strength accounting for LTB can be found in the AISC 2005 Specification^[A.12.5-1] and BS 5400-3^[A.12.5-2].

A.12.6.2.7 Bending strength check

Non-circular prismatic members subjected to bending moments, M_u , should satisfy Equation (A.12.6-37):

$$M_u \leq M_b / \gamma_{R,Pb} \tag{A.12.6-37}$$

where

M_u is M_{uy} or M_{uz} , the bending moment due to factored actions about member y- and z-axes, respectively;

M_b is the representative bending moment strength, determined from A.12.6.2.5 and A.12.6.2.6;

$\gamma_{R,Pb}$ is the partial resistance factor for bending, $\gamma_{R,Pb} = 1,1$.

A.12.6.3 Non-circular prismatic member combined strength checks

A.12.6.3.1 General

There are two different assessment approaches for the strength of non-circular prismatic members subjected to combined axial forces and bending moments:

- a) the interaction equation approach (see A.12.6.3.2), which is applicable to all member classifications;
- b) the plastic interaction surface approach (see A.12.6.3.3), which is applicable to members in class 1 and class 2.

A.12.6.3.2 Interaction equation approach

Each non-circular prismatic structural member should satisfy the following conditions in Equations (A.12.6-38) to (A.12.6-40) at all cross-sections along its length. When the shear due to factored actions is greater than 60 % of the shear strength, the bending moment strength should be reduced parabolically to zero when the shear equals the shear strength (P_v in A.12.6.3.4).

Local strength check (for all members) is as given in Equation (A.12.6-38):

$$\frac{\gamma_{R,Pa} P_u}{P_{pls}} + \left[\left(\frac{\gamma_{R,Pb} M_{uey}}{M_{by}} \right)^\eta + \left(\frac{\gamma_{R,Pb} M_{uez}}{M_{bz}} \right)^\eta \right]^{1/\eta} \leq 1,0 \tag{A.12.6-38}$$

Beam-column check (for members subject to axial compression) is as given in Equation (A.12.6-39) or Equation (A.12.6-40):

— if $\gamma_{R,Pa} P_u / P_p > 0,2$, then

$$\frac{\gamma_{R,Pa}P_u}{P_p} + \frac{8}{9} \left[\left(\frac{\gamma_{R,Pb}M_{uay}}{M_{by}} \right)^\eta + \left(\frac{\gamma_{R,Pb}M_{uaz}}{M_{bz}} \right)^\eta \right]^{\frac{1}{\eta}} \leq 1,0 \quad (\text{after AISC}^{[A.12.5-1]}, \text{Equation H1-1a}) \quad (\text{A.12.6-39})$$

— if $\gamma_{R,Pa}P_u/P_p \leq 0,2$, then

$$\frac{\gamma_{R,Pa}P_u}{2P_p} + \left[\left(\frac{\gamma_{R,Pb}M_{uay}}{M_{by}} \right)^\eta + \left(\frac{\gamma_{R,Pb}M_{uaz}}{M_{bz}} \right)^\eta \right]^{\frac{1}{\eta}} \leq 1,0 \quad (\text{after AISC}^{[A.12.5-1]}, \text{Equation H1-1b}) \quad (\text{A.12.6-40})$$

where

P_u is the applied axial force in a member due to factored actions, determined in an analysis that includes P- Δ effects (see A.12.4);

P_{pls} is the representative local axial strength of a non-circular prismatic member where

$P_{pls} = P_t$ for members in tension, as defined A.12.6.2.2,

$P_{pls} = P_{pl}$ for members in compression, as defined A.12.6.2.3;

P_p is the representative axial strength of a non-circular prismatic member where

$P_p = P_n$ for members in compression, as defined A.12.6.2.4;

M_{uey} is the corrected bending moment due to factored actions about the member y-axis from A.12.4;

M_{uez} is the corrected bending moment due to factored actions about the member z-axis from A.12.4;

M_{uay} is the amplified bending moment due to factored actions about the member y-axis from A.12.4;

M_{uaz} is the amplified bending moment due to factored actions about the member z-axis from A.12.4;

M_{by} is the representative bending moment strength about the member y-axis, as defined in A.12.6.2.5 or A.12.6.2.6

When the shear due to factored actions is greater than 60 % of the shear strength, the bending moment strength should be reduced parabolically to zero when the shear equals the shear strength (P_{vz} in A.12.6.3.4). For a more detailed description of the method see Eurocode 3^[A.12.6-3];

M_{bz} is the representative bending moment strength about the member z-axis, as defined in A.12.6.2.5 or A.12.6.2.6

When the shear due to factored actions is greater than 60% of the shear strength, the bending moment strength should be reduced parabolically to zero when the shear equals the shear strength (P_{vy} in A.12.6.3.4). For a more detailed description of the method see Eurocode 3^[A.12.6-3];

$\gamma_{R,Pb}$ is the partial resistance factor for bending strength, $\gamma_{R,Pb} = 1,1$;

$\gamma_{R,Pa}$ is the partial resistance factor for axial strength where

$\gamma_{R,Pa} = \gamma_{R,Pt}$ for axial tensile strength, $\gamma_{R,Pa} = 1,05$ in Equations (A.12.6-38, A.12.6-39 and A.12.6-40),

$\gamma_{R,Pa} = \gamma_{R,Pcl}$ for axial compressive strength, $\gamma_{R,Pa} = 1,1$ in Equation (A.12.6-38),

$\gamma_{R,Pa} = \gamma_{R,Pc}$ for axial compressive strength, $\gamma_{R,Pa} = 1,1$ in Equations (A.12.6-39) and (A.12.6-40);

η is the exponent for biaxial bending, a constant dependent on the member cross-section geometry, determined as follows:

- for purely circular tubular members $\eta = 2,0$;
- for solid or hollow rectangular sections $\eta = 5/3$;
- for doubly symmetric open section members $\eta = 1,0$;
- for all geometries, a conservative value of $\eta = 1,0$ may be used.

Annex F presents an approach to determining the value of η by manual calculation. The following mapping of the variables should be applied.

- a) M'_{uey}, M'_{uez} should be set to the applicable of M_{uey}, M_{uez} or M_{uay}, M_{uaz} , respectively, as described above.
- b) M_{ny}, M_{nz} should be set to M_{by}, M_{bz} , respectively.

A.12.6.3.3 Interaction surface approach

In the interaction surface approach, the assessor develops a plastic strength interaction surface in terms of the axial strength and biaxial moment strengths. The interaction surface can be based on Dyer^[A.12.6-2] and can be used for the strength checks. The approach is based on axial force applied at the “centre of squash”, which is defined as the location at which the axial force produces no moment on the fully plastic section.

IMPORTANT — The assessor should be aware that the sign of the moment is crucially important for sections without material or geometric symmetry. The sign convention should, therefore, be observed with care.

NOTE A common case where errors in sign can be introduced is when taking the results of a computer analysis and applying them to a series of parametric equations that can have a different axis convention.

A measure of the interaction ratio can, then, be obtained as the ratio between the vector length from the functional origin to the member forces, and the vector length from the functional origin to the nearest point on the surface. The functional origin is the force point associated with the functional actions in the absence of environmental actions.

Annex F provides, by way of example, conservative interaction equations and curves for generic families of chord cross-sections based on plastic strengths P_y, M_{py} , and M_{pz} . The resistance factors should be introduced by the assessor. This is achieved by the definitions as given in Equations (A.12.6-41) to (A.12.6-43):

$$P_y = P_{pl}/\gamma_{R,Pa} \quad \text{strength check (for all members)} \quad (\text{A.12.6-41})$$

$$\text{or } P_y = P_p/\gamma_{R,Pa} \quad \text{beam-column check (for members subject to axial compression)}$$

$$M_{py} = M_{by}/\gamma_{R,Pb} \quad (\text{A.12.6-42})$$

$$M_{pz} = M_{bz}/\gamma_{R,Pb} \quad (\text{A.12.6-43})$$

where

M_{by} is the representative bending moment strength, as defined in A.12.6.3.2;

M_{bz} is the representative bending moment strength, as defined in A.12.6.3.2;

P_p is the representative axial strength of a non-circular prismatic member, as defined in A.12.6.3.2;

P_{pls} is the representative local axial strength of a non-circular prismatic member, as defined in A.12.6.3.2

$\gamma_{R,Pb}$ is the partial resistance factor for bending strength, $\gamma_{R,Pb} = 1,1$;

$\gamma_{R,Pa}$ is the partial resistance factor for axial strength where

$\gamma_{R,Pa} = \gamma_{R,Pt}$ for axial tensile strength, $\gamma_{R,Pt} = 1,05$;

$\gamma_{R,Pa} = \gamma_{R,Pc}$ for axial compressive strength, $\gamma_{R,Pc} = 1,1$;

$\gamma_{R,Pa} = \gamma_{R,Pcl}$ for local strength, $\gamma_{R,Pcl} = 1,1$.

For the strength check, the applied member forces (P , M_y , M_z in Annex F) should be P_u , M_{uey} , M_{uez} as defined in A.12.6.3.2.

For the beam-column check, the applied member forces (P , M_y , M_z in Annex F) should be P_u , M_{uay} , M_{uaz} as defined in A.12.6.3.2.

A.12.6.3.4 Beam shear

Non-circular prismatic members subjected to beam shear forces due to factored actions should satisfy Equations (A.12.6-44) and (A.12.6-45):

$$V_y \leq P_{vy} / \gamma_{R,Pv} \quad (\text{A.12.6-44})$$

$$V_z \leq P_{vz} / \gamma_{R,Pv} \quad (\text{A.12.6-45})$$

where

V_y , V_z is the beam shear due to factored actions in the local y- and z-direction, respectively;

P_{vy} , P_{vz} is the representative shear strength in the local y- and z-directions, respectively, as given in Equation (A.12.6-46):

$$P_{vy}, P_{vz} = A_v F_{ymin} / \sqrt{3} \quad (\text{A.12.6-46})$$

A_v is the effective shear area in the direction being considered; see Table A.12.6-1;

$\gamma_{R,Pv}$ is the partial resistance factor for torsional and beam shear strengths, $\gamma_{R,Pv} = 1,1$.

Table A.12.6-1 — Effective shear area for various cross-sections

Section	Effective shear area, A_v
Rolled I, H and channel sections, load parallel to web	$t D_s$
Welded I sections, load parallel to web	$t d$
Rectangular hollow sections, load parallel to webs	$A D_s / (D_s + B_s)$
Welded box sections, load parallel to web	$2 t d$
Rolled Tee-sections, load parallel to web	$t D_s$
Welded Tee-sections, load parallel to web	$t (D_s - T)$
Circular hollow sections	$0,5 A$
Solid bars and plates	$0,9 A$
Closed sections with inclined plates	$0,9 \sum [\cos(\theta_i) A_{oi}]$
<p>T is the flange thickness of a welded T-section. t is the web thickness. D_s is the overall depth of cross-section. d is the web depth; for rolled sections measured with respect to root radii, for welded sections measured between inside faces of flanges. B_s is the overall breadth of cross-section. A is the area of cross-section. A_{oi} is the area of rectilinear component i. θ_i is the angle between the shear force direction being considered and the larger dimension of the cross-section of component i.</p>	

A.12.6.3.5 Torsional shear

Closed-section non-circular prismatic members subjected to torsional shear forces due to factored actions should satisfy Equation (A.12.6-47):

$$T_u \leq T_{vp} / \gamma_{R,Pv} \tag{A.12.6-47}$$

where

T_u is the torsional moment due to factored actions;

T_{vp} is the representative torsional strength of the non-circular prismatic member as given in Equation (A.12.6-48):

$$T_{vp} = I_{pp} F_{ymin} / (r \sqrt{3}) \tag{A.12.6-48}$$

I_{pp} is the polar moment of inertia of the non-circular prismatic member;

r_t is the maximum distance from centroid to an extreme fibre;

$\gamma_{R,Pv}$ is the partial resistance factor for torsional and beam shear strengths, $\gamma_{R,Pv} = 1,1$.

Open-section non-circular prismatic members subjected to torsional shear forces should be checked as appropriate.

A.12.7 Assessment of joints

Joints should be assessed when the site conditions (metocean combinations, eccentric spudcan loading, etc.) fall outside the limits that are normally assessed by the RCS.

The designer can make joint strengths available to the assessor. When the supplied axial joint strength is less than the member strength, the supplied joint strength should be used in lieu of the member axial strength in member strength checks.

If it is considered necessary to evaluate joint strength, the resistance of tubular joints can be assessed in accordance with ISO 19902:2007, 24.9.2.2.2 and A.24.9.2.2.2 (Connections), and that of non-tubular joints by rational analysis. The internal forces (action effects) due to factored actions should be determined in accordance with 8.8, rather than using ISO 19902 and ISO 19901-3.

NOTE The intent of the joint check is to ensure that the joint is strong enough to resist the internal forces due to factored actions. The joint strength is not required to meet or exceed the full member strength. Guidance on non-tubular joint strength can be found in other provisions of ISO 19902 and ISO 19901-3.

A.13 Acceptance checks

No guidance is offered.

Annex B (normative)

Summary of partial action and partial resistance factors

Symbol	Description	Factor	Subclause(s)
$\gamma_{f,D}$	partial action factor applied to the inertial actions D_e due to dynamic response, in combination with $\gamma_{f,E}$	1,0	8.8.1.1 to 8.8.1.4
$\gamma_{f,G}$	partial action factor applied to the fixed actions G_F	1,0	8.8.1
$\gamma_{f,V}$	partial action factor applied to the actions due to variable load G_V	1,0	8.8.1
$\gamma_{f,E}$	partial action factor applied to the metocean or earthquake actions		8.8.1.1 to 8.8.1.4
	when applied to deterministic ULS storm action E_e (used with 50 year independent extreme values)	1,15	8.8.1.2
	when applied to the deterministic ULS storm action E_e (used with 100 year joint probability metocean data)	1,25	8.8.1.2
	when applied to the stochastic ULS storm actions E_e using factored metocean parameters determined in accordance with A.10.5.3.2 ^a	1,0	8.8.1.3
	when applied to the inertial action induced by the ELE ground motions in earthquake analysis	0,9	8.8.1.4.1
	when applied to the inertial action induced by the ALE ground motions in earthquake analysis	1,0	8.8.1.4.2
$\gamma_{R,PRE}$	partial resistance factor for preload	1,1	A.9.3.6.2
$\gamma_{R,Hfc}$	partial resistance factor for horizontal foundation capacity for effective stress (sand/drained)	1,25	A.9.3.6.2
	partial resistance factor for horizontal foundation capacity for total stress (clay/undrained)	1,56	A.9.3.6.2
$\gamma_{R,VH}$	partial resistance factor for vertical-horizontal foundation bearing capacity	1,1	A.9.3.6.4
$\gamma_{R,Tb}$	partial resistance factor for bending strength of a tubular ^b	1,05	A.12.5
$\gamma_{R,Tc}$	partial resistance factor for axial compressive strength of a tubular ^b	1,15	A.12.5
$\gamma_{R,Tt}$	partial resistance factor for axial tensile strength of a tubular ^b	1,05	A.12.5
$\gamma_{R,Tv}$	partial resistance factor for torsional and beam shear strengths of a tubular ^b	1,05	A.12.5
$\gamma_{R,Pb}$	partial resistance factor for bending strength prismatic of a non-circular prismatic member ^b	1,1	A.12.6
$\gamma_{R,Pc}$	partial resistance factor for axial compressive strength of a non-circular prismatic member ^b	1,1	A.12.6
$\gamma_{R,Pcl}$	partial resistance factor for local axial compressive strength of a non-circular prismatic member ^b	1,1	A.12.6
$\gamma_{R,Pt}$	partial resistance factor for axial tensile strength of a non-circular prismatic member ^b	1,05	A.12.6
$\gamma_{R,Pv}$	partial resistance factor for torsional and beam shear strengths of a non-circular prismatic member	1,1	A.12.6

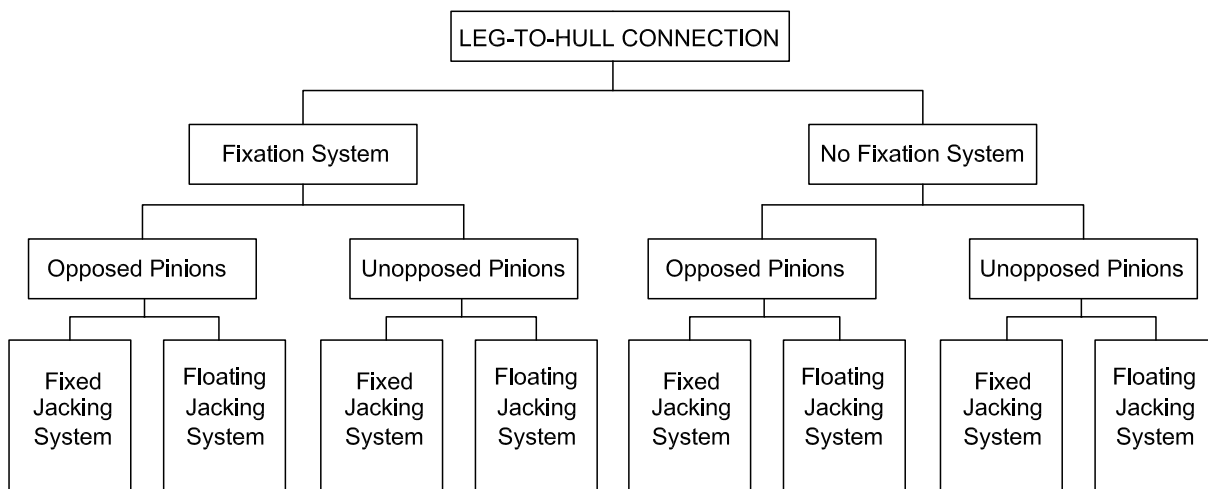
Symbol	Description	Factor	Subclause(s)
$\gamma_{R,S}$	partial resistance factor for spudcan strength	1,15	13.4
$\gamma_{R,H}$	partial resistance factor for holding system	1,15	13.5
$\gamma_{R,OTM}$	partial resistance factor for stabilizing moment	1,05	13.8
NOTE Values given in this table are normative. The reference subclauses provide the methods of application and the factors are specifically tied to the calculation methodologies given in each reference subclause.			
<p>^a The metocean partial factors used in the quasi-static stochastic analysis are determined through an iterative procedure. The procedure involves factoring the metocean parameters (wave height, current velocity and wind) until the partial-factored quasi-static stochastic force matches the action-factored quasi-static deterministic force. The start point for the iteration can be taken as $\sqrt{\gamma_{f,E}}$.</p> <p>^b The structural resistance factors for tubular members given in Clause 12 are based on an independent interpretation of the theoretical values derived from the data used in the calibration of API RP 2A LRFD, 1st edition, to API RP 2A, 15th edition, and the data used in the development of the ISO 19902 tubular members strength formulations. The values for non-tubular prismatic members were based on AISC ^[A.12.5-1], which changed its equivalent resistance factor from 1,18 to 1,1 between the 1986 and 2005 editions because a reassessment of the applicable data resulted in an effective reduction in the coefficient of variation.</p>			

Annex C (informative)

Additional information on structural modelling and response analysis

C.1 Guidance on 8.5 — Modelling the leg-to-hull connections

The potential leg-to-hull connection component arrangements are shown in Figure C.1-1, which also gives examples of jack-ups designs in each category.



Examples of jack-ups in each category

FandG - L780 II - JU 2000 - Alpha 350 - Super M2 - Universal M class	GustoMSC - CJ36 - CJ46 - CJ50 (old) - CJ54 - CJ62 - CJ70	NONE	NONE	Baker Marine - Pacific 375	CFEM - 2005 - 2600	Baker Marine - Freedom class - 350 - 300 - 250 - 200 - 150	NONE
GustoMSC - CJ 46 - CJ 50 (new)				LeTourneau - Workhorse - Tarzan	Levingston - 111C		
Hitachi - Giant class				Modec - 300C - 400		GustoMSC - Gusto designs	
Keppel FELS - KFELS Mod V - KFELS Mod VI - A Class - B Class - N Class						LeTourneau - 53 - 84 - 82-SD-C - 116C - Super 116 - Super 116E - Super 300 - Gorilla	
LeTourneau - Super GORILLA - Super GORILLA XL - Jaguar 250-C							

Figure C.1-1 — Sample leg-to-hull connection component combinations

C.2 Guidance on A.10.5.3.4 — Methods for determining the MPME

C.2.1 Guidance on the first method of Table A.10.5-1 — Fitting Weibull distributions to the results of a number of time domain simulations to determine responses at the required probability level and average the results

This procedure, outlined in Steps 1 to 7 below, requires several suitable length time domain simulations for each response of interest. The input sea state simulation should be checked for Gaussianity. Guidance is given in Table A.7.3-3. For each simulation record, the procedure for computing the MPME comprises the following steps. The final MPME value is taken as the average over all of the simulations conducted.

— **Step 1:**

The response time history, $R(t)$, is first analysed to calculate the mean, μ_R , as given in Equation (C.2.1-1):

$$\mu_R = \frac{\sum_{i=1}^n R(t_i)}{n} \quad (\text{C.2.1-1})$$

where

$R(t_i)$ is the time history of the response of interest;

t_i is the time point i ;

n is the number of useable time points in simulation (discounting the run-in).

— **Step 2:**

The individual response maxima in the simulations are next extracted according to the following criteria:

A maximum occurs at t_i if Equations (C.2.1-2) apply:

$$R(t_{i-1}) < R(t_i) \text{ and } R(t_{i+1}) \leq R(t_i) \quad (\text{C.2.1-2})$$

Suppose N_{\max} maxima are found in the extraction.

— **Step 3:**

From the N_{\max} response maxima, the mean of the signal, μ_R , is subtracted and the resulting maxima M_k , where k varies from 1 to N_{\max} , are ranked into 20 blocks having mid-points in ascending order. The blocks all have the same width; the upper bound of block 20 is taken as 1,01 times the largest value, and the lower bound of the first block is set to zero. Any maxima with a value less than zero are discarded. The blocks are numbered in ascending order from $q = 1$ to 20, and are defined by their midpoint value M_q^* and the probability of non-exceedance of that value F_q . A distribution of the observed maxima is then found, using for each block the Gumbel plotting position in order to obtain the best possible description of the distribution for large values of M . If the number of maxima in each block, q , is n_q , the cumulative probability F_q to plot against the mid-point for block q is then as given in Equation (C.2.1-3):

$$F_q = \left[\left(1 + \sum_{j=0}^{q-1} n_j \right) \left(\sum_{j=0}^q n_j \right) \right]^{0,5} / (N_{\max} + 1) \quad (\text{C.2.1-3})$$

where n_0 is equal to the number of negative maxima peaks (the number of points not fitting into the 20 blocks).

— **Step 4 a):**

A Weibull distribution is fitted [see Steps 4 b) to 4 d)] through the cumulative distribution of the blocks of observed maxima as defined under Step 3 [this is done in accordance with Steps 4 b) to 4 d)]. The 3-parameter Weibull cumulative distribution function is defined as given in Equation (C.2.1-4):

$$F(M^*; \alpha, \beta, \gamma) = 1 - \exp \left[- \left(\frac{M^* - \gamma}{\alpha} \right)^\beta \right] \quad (\text{C.2.1-4})$$

where

$F(M^*; \alpha, \beta, \gamma)$ is the probability of non-exceedance of the value M^* where

α is the scale parameter,

β is the slope parameter,

γ is the threshold parameter;

$$\alpha, \beta, (M^* - \gamma) > 0, 0$$

— **Step 4 b):**

Only data blocks with a probability of non-exceedance greater than a threshold value of 0,2 are used to fit the Weibull distribution, i.e. only the blocks for which Equation (C.2.1-5) applies:

$$F_q > 0,2 \quad (\text{C.2.1-5})$$

Notice that M_q^* are in ascending order.

— **Step 4 c):**

For each of these blocks, q , the deviations, δ_q , of the observed probability from the corresponding probability of the Weibull cumulative distribution function, F , (transformed to Weibull scales) are calculated as given in Equation (C.2.1-6):

$$\delta_q = \ln\{-\ln[1 - F(M_q^*; \alpha, \beta, \gamma)]\} - \beta [\ln(M_q^* - \gamma) - \ln(\alpha)] \quad (\text{C.2.1-6})$$

— **Step 4 d):**

The parameters α, β, γ are now estimated by a non-linear least squares technique such that the summation as given in Equation (C.2.1-7) is minimized:

$$\sum_{q=x}^{20} \delta_q^2$$

where x is the value of q for which $F_q > 0,2$. (C.2.1-7)

The procedure may be based on a Levenberg-Marquardt algorithm, using the parameters of a 2-parameter Weibull distribution (found by the maximum likelihood method) as initial estimates.

— **Step 5:**

The MPM value, M_{MPM} , is found as the value of M for which Equation (C.2.1-8) applies:

$$F(M^*; \alpha, \beta, \gamma) = 1 - \frac{1}{\left\{ N_{\text{max}} \frac{T_{3\text{h}}}{T_{\text{sim}}} \right\}} \quad (\text{C.2.1-8})$$

where

$T_{3\text{h}}$ is 3 hours;

T_{sim} is the simulation duration.

— **Step 6:**

The corresponding MPME value, R_{MPME} is then found as given in Equation (C.2.1-9):

$$R_{\text{MPME}} = \mu_R + M_{\text{MPM}} \quad (\text{C.2.1-9})$$

where

μ_R is the mean value of $R(t)$ established in Step 1;

M_{MPM} is the MPME value (excluding the mean) established in Step 5.

— **Step 7:**

The procedure is repeated for each required response parameter.

C.2.2 Guidance on the second method of Table A.10.5-1: Fitting Gumbel distribution to histogram of absolute maximum responses from a number of time domain simulations to determine responses at required probability level

The basic assumption of this method is that the absolute maximum values in three-hour simulations follow a Gumbel distribution as given in Equation (C.2.2-1):

$$F_{3\text{h}}(x) = \exp \left[-\exp \left(-\frac{x - \psi}{\kappa} \right) \right] \quad (\text{C.2.2-1})$$

where

$F_{3\text{h}}(x)$ is the probability that the three-hour maximum does not exceed value x ;

ψ is the location parameter;

κ is the scale parameter.

The following steps are followed for each required response parameter:

— **Step 1:**

Extract absolute maximum (and minimum) value for each of at least ten three-hour response simulations.

— **Step 2:**

Fit a Gumbel distribution through these 10 or more maxima/minima. This can be done using the maximum likelihood method, yielding ψ and κ . Alternatively, Lu *et al.*^[C.2-1] have shown that the method of moments closed form solution produces results consistent with the maximum likelihood method for values of ψ , although they showed significant variation in the values of κ . However, when calculating the MPME, with a 63 % probability of exceedance, the effects of κ approach zero, as shown in Step 3 below, and the only remaining influence is on the calculated value of ψ , as given in Equation (C.2.2-3). Therefore, the method of moments closed form solution normally can be used to calculate ψ and κ as given in Equations (C.2.2-2) and (C.2.2-3):

$$\kappa = (\sigma \sqrt{6})/\pi \tag{C.2.2-2}$$

$$\psi = \mu - 0,577 \kappa \tag{C.2.2-3}$$

where

μ is the mean of the maxima/minima;

σ is the standard deviation of the maxima/minima.

— **Step 3:**

The value R_{MPME} is found as given in Equations (C.2.2-4) and (C.2.2-5):

$$R_{MPME} = \psi - \kappa \ln \left\{ -\ln \left[F_{3h}(R_{MPME}) \right] \right\} \tag{C.2.2-4}$$

where

$$F_{3h}(R_{MPME}) = 0,37 \tag{C.2.2-5}$$

The 0,37 lower quantile is used because the extreme of recurrence of once in three hours has a probability of exceedance of 0,63 (= 1 – 0,37). In this case, it can be seen that

$$R_{MPME} = \psi$$

— **Step 4:**

The procedure of Steps 2 and 3 can similarly be applied for minima although, because of the potential error in κ , and because the standard deviation of the minima can be large by comparison to the mean, the method of moments should not be used for calculating κ and ψ .

C.2.3 Guidance on the third method of Table A.10.5-1 — Application of Winterstein's Hermite polynomial method to the results of time domain simulation(s)

For Gaussian processes, analytical results exist for the determination of the MPM values (e.g. MPM wave height is 1,86 times the significant wave height). For general non-linear, non-Gaussian, finite band-width processes, approximate methods are used to generate the probability density function of the process. The method proposed by Winterstein^[C.2-2] fits a Hermite polynomial of Gaussian processes to transform the non-linear, non-Gaussian process into a mathematically tractable probability density function. This has been further refined by Jensen^[C.2-3] for processes with large kurtosis.

This procedure requires a suitable length time domain simulation for each quantity of interest. The input sea state simulation should be checked for Gaussianity. Guidance is given in Table A.7.3-3. The calculation procedure to determine the maximum of a time series, $R(t)$, with a simulation duration T_{sim} for a three-hour exposure, T_{3h} , is as follows.

— **Step 1:**

Calculate the mean, μ , the standard deviation, σ , and the following quantities of the time series for the parameter under consideration as given in Equations (C.2.3-1) and (C.2.3-2):

$$\alpha_3 = (1/n) \sum [(R - \mu)^3] / \sigma^3 \quad (\text{C.2.3-1})$$

$$\alpha_4 = (1/n) \sum [(R - \mu)^4] / \sigma^4 \quad (\text{C.2.3-2})$$

where

α_3 is the skewness;

α_4 is the kurtosis.

When the kurtosis is less than 3,0, the approach given here is not valid and the alternative given in Reference [C.2-2] should be used.

— **Step 2:**

Construct a standardized response process, $z = (R - \mu) / \sigma$. Using this standardized process, calculate the number of zero-upcrossings, N . In lieu of an actual cycle count from the simulated time series, $N = 1\,000$ may be assumed for a three-hour simulation.

— **Step 3:**

Compute the following quantities from the characteristics of the time series for the response of interest as given in Equations (C.2.3-3) to (C.2.3-5):

$$h_3 = \alpha_3 / \left\{ 4 + 2 \sqrt{1 + 1,5(\alpha_4 - 3)} \right\} \quad (\text{C.2.3-3})$$

$$h_4 = \left\{ \sqrt{1 + 1,5(\alpha_4 - 3)} - 1 \right\} / 18 \quad (\text{C.2.3-4})$$

$$K = \left(1 + 2h_3^2 + 6h_4^2 \right)^{-1/2} \quad (\text{C.2.3-5})$$

It is necessary to seek a more accurate result by determining a solution for C_1 , C_2 and C_3 from Equations (C.2.3-6) to (C.2.3-8):

$$\sigma^2 = C_1^2 + 6C_1C_3 + 2C_2^2 + 15C_3^2 \quad (\text{C.2.3-6})$$

$$\sigma^3\alpha_3 = C_2(6C_1^2 + 8C_2^2 + 72C_1C_3 + 270C_3^2) \quad (\text{C.2.3-7})$$

$$\begin{aligned} \sigma^4\alpha_4 = & 60C_2^4 + 3C_1^4 + 10\,395C_3^4 + 60C_1^2C_2^2 + 4\,500C_2^2C_3^2 + 630C_1^2C_3^2 + \dots \\ & \dots + 936C_1C_2^2C_3 + 3\,780C_1C_3^3 + 60C_1^3C_3 \end{aligned} \quad (\text{C.2.3-8})$$

using as initial guesses the values given in Equations (C.2.3-9) to (C.2.3-11):

$$C_1 = \sigma K(1 - 3h_4) \quad (\text{C.2.3-9})$$

$$C_2 = \sigma Kh_3 \quad (\text{C.2.3-10})$$

$$C_3 = \sigma Kh_4 \quad (\text{C.2.3-11})$$

where

σ is obtained from Step 1;

K , h_3 and h_4 are obtained from Equations (C.2.3-3) to (C.2.3-4).

Following the solution for C_1 , C_2 and C_3 , the values for K , h_3 and h_4 are recomputed as given in Equations (C.2.3-12) to (C.2.3-14):

$$K = (C_1 + 3C_3)/\sigma \quad (\text{C.2.3-12})$$

$$h_3 = C_2/(\sigma K) \quad (\text{C.2.3-13})$$

$$h_4 = C_3/(\sigma K) \quad (\text{C.2.3-14})$$

— **Step 4:**

The most probable value, U , of the transformed process is computed as given in Equation (C.2.3-15):

$$U = \sqrt{2 \ln \left(N \cdot \frac{T_{3h}}{T_{sim}} \right)} \quad (\text{C.2.3-15})$$

where

U is a Gaussian process of zero mean, unit variance;

T_{3h} is three hours;

T_{sim} is the simulation duration, expressed in hours.

— **Step 5:**

The most probable maximum, transformed back to the standardized variable, z , is then as given in Equations (C.2.3-16):

$$z_{MPM} = K [U + h_3(U^2 - 1) + h_4(U^3 - 3U)] \quad (\text{C.2.3-16})$$

— **Step 6:**

Finally, the MPME in the required three hour exposure period for the response under consideration, can be computed from Equation (C.2.3-17):

$$R_{MPME} = \mu + \sigma z_{MPM} \quad (\text{C.2.3-17})$$

C.2.4 Guidance on the fourth method of Table A.10.5-1: Application of drag-inertia method to determine the base shear and overturning moment DAF from time domain simulation

The method, based on Reference [C.2-4], may be used to determine $K_{DAF,RANDOM}$ used to compute the inertial loadset for a two-stage deterministic storm analysis (see 10.5.2). The method combines two components of the total dynamic response, namely the static and inertial parts. The inertial part is computed as the difference between the total dynamic and static responses and should not be confused with the response to inertial wave loading. The method requires a determination of the response of the jack-up for four conditions. In all four cases, the storm simulation (random seed) should be identical, but with different components of the loading and/or response simulated. The responses considered are usually total wave and

current base shear and total wave and current overturning moment, for computing the base shear and overturning moment DAFs, respectively.

The four cases being simulated are full dynamic response, full static response, static response to inertia only wave loading (setting $C_d = 0$) and static response to drag only loading (setting $C_m = 0$). From these the inertial response is obtained as the full dynamic response minus the full static response. The means and standard deviations of the response are extracted from the time domain responses and the DAFs computed as illustrated in Figure C.2.4-1.

The drag-inertia method given here includes a final step to scale the DAF based on the period ratio T_n/T_p . This step is included to ensure that the DAF values are not underestimated for cases where T_n approaches T_p ; see Perry and Mobbs^[C.2-5]. The equation for the scaling factor is shown in Figure C.2.4-1 and is illustrated graphically in Figure C.2.4-2.

This method should not be used to compute MPME values for use in a one-stage stochastic analysis. It should be used only in a two-stage analysis when the foundation is modelled as either pinned or based on linearized stiffness in the DAF calculation.

Further details on the background to, and limitations of, this method can be found in ISO/TR 19905-2.

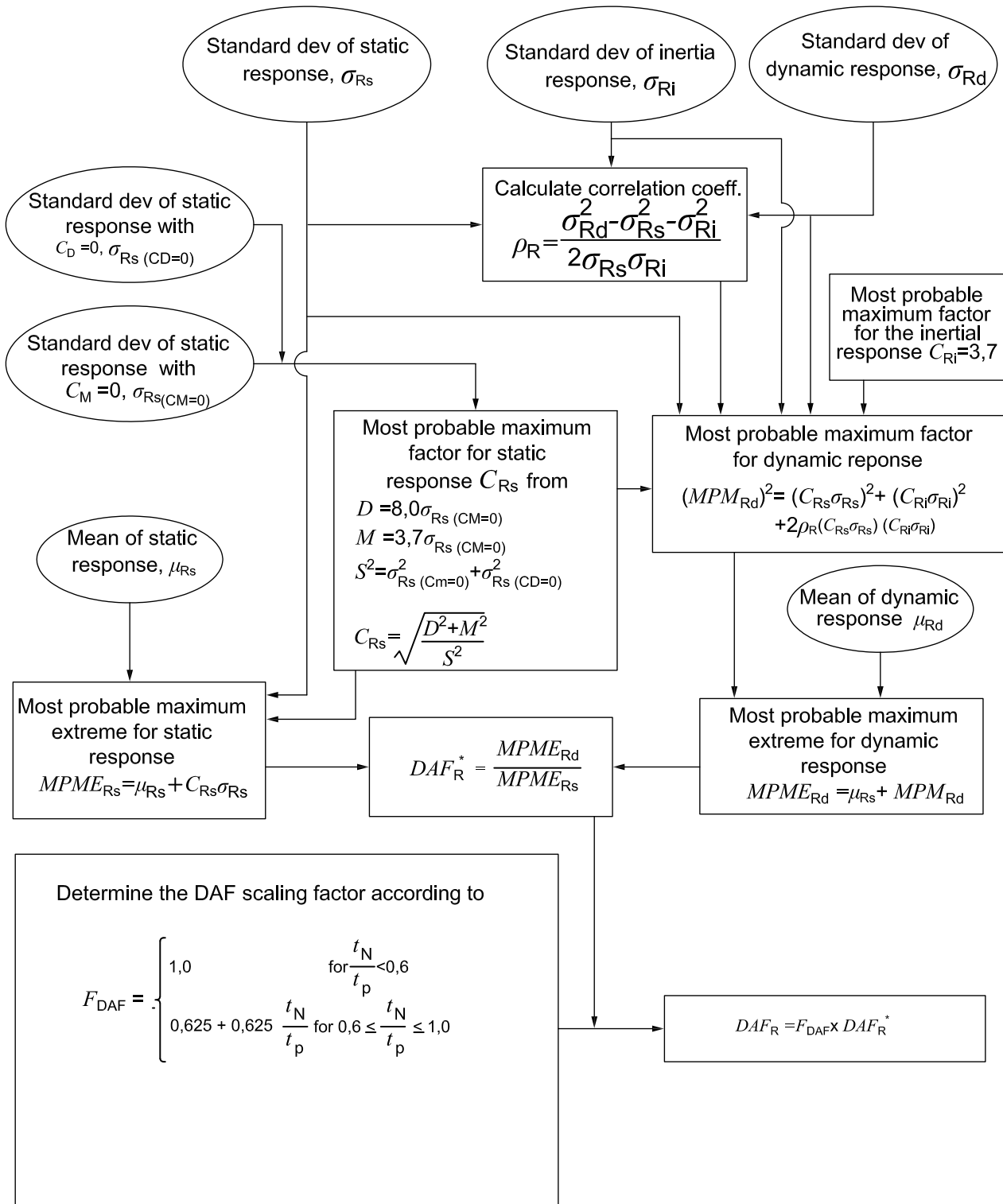


Figure C.2.4-1 — The drag-inertia method including DAF scaling factor

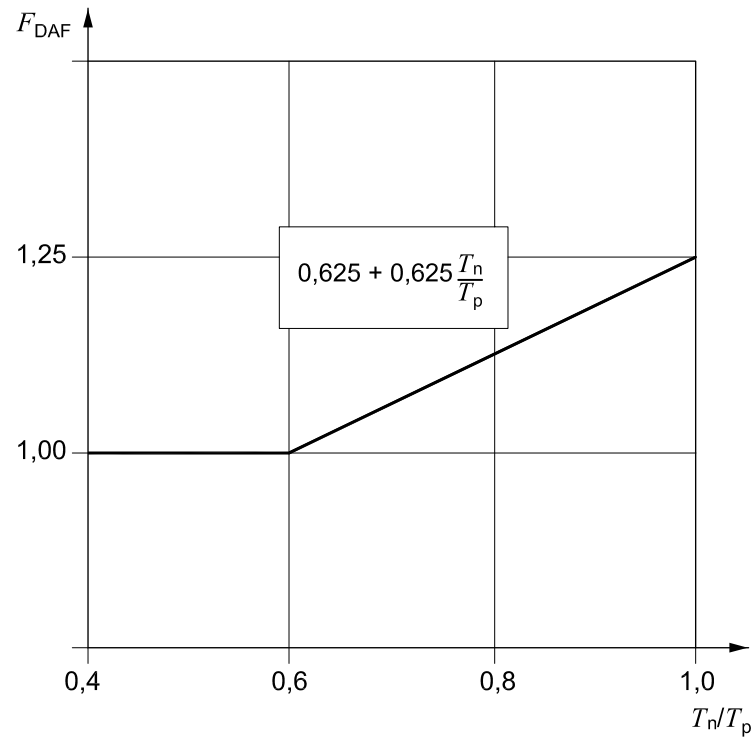


Figure C.2.4-2 — Graphical representation of DAF scaling factor, F_{DAF} , applied in the drag-inertia method

Annex D (informative)

Foundations — Recommendations for the acquisition of site-specific geotechnical data

This annex, based on a report by the InSafeJIP^[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 optimize 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 that can 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 minimize 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; see 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 used for work-over or as installation support facilities throughout the field life, then it is necessary to give due consideration to the positioning of these units and the implication of the seabed depressions and zones of disturbed soil (footprints) caused during spudcan installation on 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.

Although there can be exceptions, it is generally necessary to acquire geotechnical data to a depth below sea floor of either 30 m, or the anticipated spudcan penetration depth plus 1,5 spudcan diameters, whichever is the greater.

It is also recognized that conducting an optimally designed offshore work-scope is not always possible so that a compromise solution is necessary. Such factors 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.

Tables D.1 through D.3 list various site conditions and provide recommendations for offshore geotechnical site investigation works that can be required to conduct a jack-up foundation site-specific assessment for both “open” and “work-over” sites. “Open sites” refer to sites where no jack-up has previously operated, whereas “work-over” sites are sites at which jack-ups have previously been installed.

At work-over sites, the ground is likely to have been disturbed and craters, or “footprints”, left at the seabed at previously installed spudcan locations. These operations are likely to have modified the soil properties and such ground modification should be considered during the assessment of future jack-up installation operations. Spudcan-footprint interaction issues are not covered in this annex.

Tables D.1 to D.3 provide guidance on the number and positioning of geotechnical boreholes (purely sampling or combined or “composite” sampling and down-hole testing) and continuous piezocone penetration tests for a range of circumstances. Refer to A.6.5 and Reference [D.1-1] for further comment on soil sampling and cone penetration test data acquisition. Soil laboratory testing requirements and specifications are discussed elsewhere, A.6.5 and Reference [D.1-1].

These recommendations should be used only for guidance and do not imply any legislative requirements, responsibilities or guarantee of applicability.

Table D.1 — Geotechnical work scope for open sites for simple geological conditions

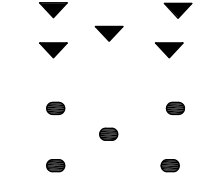
Programme type	Geological setting	Site conditions	Minimum suggested site investigation work scope ^a	Illustrated views
1S	Simple	Regional, local geology and near surface conditions reasonably well understood. Site conditions suitable for jack-up rig (JU) operations. High-quality geophysical data available and sub-bottom profiling data tied-back to a geotechnical borehole(s) and/or local JU installation sites. Mature JU operating province where foundation issues are not expected and with laterally continuous ground conditions. Desk top study corroborates geophysical data. Adverse foundation performance risk extremely remote and any potential risk is expected to be manageable	Acquisition of site-specific geotechnical data might not be required. ^a	not applicable
2S	Simple	As 1S above but with a layer of soft sediments over a hard layer of known geology, where it is expected that the spudcans are founded on the hard interface beneath the soft sediments. Formation present below the soft/hard interface known to be competent and able to safely support spudcans. The ground conditions are known to be laterally continuous and the interface between soft to harder sediments does not undulate adversely.	Seabed piezocone tests or gravity cores may be used to confirm the absence of potentially adverse layering within the soft upper sediments and to tag the hard layer. If data proves the soil conditions are not as expected, then deeper piezocone tests and/or soil boring(s) may be required to investigate and confirm the soil conditions and identify any variability. If potentially adverse conditions for JU foundations are present, consider increasing the geotechnical investigation scope of work. ^a	 <p>or combinations of both across the area; be aware that this is example layout only</p>

Table D.1 (continued)

Programme type	Geological setting	Site conditions	Minimum suggested site investigation work scope ^a	Illustrated views
3S	Simple	Regional, local geology and near surface conditions reasonably well understood and suitable for JU operations. High-quality geophysical data available and sub-bottom profiling data tied back to a geotechnical borehole/s and/or local JU installation sites. Soils expected to be laterally continuous. Desk top study corroborates geophysical data. Knowledge of regionally successful JU performance and local geotechnical borehole data available. Unlikely adverse foundation performance risk	Continuous sampling borehole or continuous seabed piezocone test from 0 to TD placed in the centre of the JU footprint or at one spudcan location. Composite borehole may be acceptable if ground conditions are simple and well defined, and proved to be as expected. Data gaps are kept to a minimum (target gaps at < 0,2 m). If data gaps > 0,2 m, or there are concerns regarding the suitability of the ground for jack-up operations, then consider additional adjacent borehole(s) with downhole piezocone tests (as opposed to seabed piezocone tests if unable to reach TD), or sampling intervals conducted over data gaps of the previously conducted borehole. If concerns remain regarding the suitability of ground conditions for the JU operations, then perform additional geotechnical site investigation. ^a	
4S	Simple	Regional and local geology reasonably well understood and near surface conditions expected to be continuous and suitable for JU operations. High-quality geophysical survey data available without sub-bottom profiler data tie lines. Desk top study correlates with geophysical data. No local geotechnical data or knowledge of successful JU performance regionally. Foundation performance risk considered unlikely	One continuous sample borehole and an adjacent piezocone test from 0 to TD within the JU footprint, or combination of composite and/or continuous sampling and piezocone test boreholes at spudcan centres, so that sufficient data are available to define the ground model. If data illustrate soil conditions suitable for JU operations and the geophysics confirms stratigraphic continuity across the site, then no further geotechnical investigation is required. If variations occur, additional data acquisition should be considered ^a	
<p>Key</p> <ul style="list-style-type: none"> ● gravity piston corer ⊕ composite borehole (downhole piezocone tests and sampling) ● continuous sampling borehole ▼ shallow seabed piezocone test ▲ continuous piezocone test 				
<p>NOTE 1 If appropriate, it is advisable that the jack-up geotechnical SI be conducted in consultation with the field development teams in order to optimize data acquisition.</p>				
<p>NOTE 2 Target depth (TD) is the greater of either 30 m or 1,5 times the spudcan diameters beneath the calculated spudcan tip penetration depth at the maximum preload.</p>				
<p>NOTE 3 At minimum, the ground model is generated to TD, with adequate data acquired for reliable definition of the model.</p>				
<p>^a The requirement for and specification of the geotechnical site investigation workscope should always be discussed and agreed with a suitable qualified and experienced offshore geotechnical engineer(s).</p>				

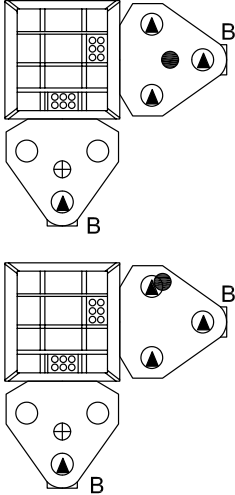
Table D.2 — Geotechnical work scope for open sites for complex/very complex geological conditions

Programme type	Geological setting	Site conditions	Minimum suggested site investigation work scope ^a	Illustrated views
1C	Complex	Regional and local geology reasonably understood without specific details of near-surface ground conditions. Desk top study and geophysical survey data ambiguous and suggests that near-surface ground conditions are likely to be variable across the site and potential for foundation performance risk recognised. No knowledge of successful JU performance locally available and potential for adverse foundation performance is recognized.	One continuous sampling borehole and one adjacent continuous piezocone test at one spudcan location and piezocone tests from 0 to TD at each of the other two spudcan locations, or continuous piezocone tests at one spudcan location with continuous sampling boreholes at the others from 0 to TD ^a	
1VC	Very complex	Regional and local geological data available without specific details of near-surface ground conditions at the site. The desk top study and geophysical survey data suggest near-surface ground conditions are variable across the site. The potential for JU foundation performance risk is identified perhaps with knowledge of adverse JU foundation performance locally.	Continuous sampling and adjacent piezocone tests at all spudcan locations from 0 to TD. Or as above with centrally located continuous sampling and/or piezocone test borehole to TD ^a	
Key ● continuous sampling borehole ▲ continuous piezocone test				
NOTE 1 If appropriate, it is advisable that the jack-up geotechnical SI be conducted in consultation with the field development teams in order to optimize data acquisition.				
NOTE 2 Target depth (TD) is the greater of either 30 m or 1,5 times the spudcan diameters beneath the calculated spudcan tip penetration depth at the maximum preload.				
NOTE 3 At minimum, the ground model is generated to TD, with adequate data acquired for reliable definition of the model.				
^a The requirement for and specification of the geotechnical site investigation workscope should always be discussed and agreed with a suitable qualified and experienced offshore geotechnical engineer(s).				

Table D.3 — Geotechnical work scope for work-over sites for simple to very complex geological settings

Programme type	Geological setting	Site conditions	Minimum suggested site investigation work scope ^a	Illustrated views
1S-WO	Simple	First JU operation at the site, no existing spudcan footprints to consider. High-quality geophysical survey available with recent seabed clearance survey. Appropriate geotechnical and geophysical data acquired for fixed platform and JU installation purposes confirms suitable conditions for JU installation. Local JU operations without foundation hazards.	No additional geotechnical data acquired for JU installation ^a	not applicable
2S-WO	Simple	Repeat visit with identical JU footprint and spudcan size. Identical spudcan positions - footprint interaction issues unlikely. No previous foundation issues and previous jack-up operations not able to adversely alter the ground conditions with identical jack-up emplacement	No new geotechnical data required ^a	not applicable
3S-WO	Simple	Repeat visit with identical JU. New spudcan positions with possible spudcan-footprint issues. 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 can be required ^a	—
4S-WO	Simple	First visit of JU to this platform where units have previously operated at the site. Known ground conditions. No previous foundation issues. (Consideration of the effects of the spudcan bearing pressures on fixed structure foundations can be necessary).	It is necessary to consider spudcan-footprint interaction issues. Consideration of spudcan-footprint interaction mitigation. Survey of existing footprints advisable with seabed clearance survey. Consideration of spudcan-footprint interaction mitigation. Additional geotechnical data can be required. ^a	—
1C-WO	Complex	First visit of JU to a platform where units have not previously operated with the knowledge of local jack-up foundation performance. New unit with spudcan bearing pressures greater than those units previously operated at the site. Check adequacy of existing geotechnical data – additional site investigation can be required.	It is necessary to consider spudcan - footprint interaction issues. Survey of existing footprints advisable with seabed clearance survey. Consideration of spudcan-footprint interaction mitigation. Consideration of spudcan-footprint interaction mitigation. Additional geotechnical data can be required. ^a The previous JU operations can have modified the ground conditions where the intended spudcan installation position is being placed and this can require investigation. ^a	—

Table D.3 (continued)

Programme type	Geological setting	Site conditions	Minimum suggested site investigation work scope ^a	Illustrated views
1VC-WO	Complex	<p>First visit of JU to this platform where units have previously operated at the site but on a different side of jacket. Previous foundation issues or no knowledge of successful JU performance locally available and potential for foundation performance recognised. Regional and local geology reasonably understood without specific details of near-surface ground conditions. At sites where the desk top study and geophysical survey data suggest near-surface ground conditions are variable across the site or risk identified and knowledge of adverse JU foundation performance locally, high potential for foundation performance risk identified. New unit with spudcan bearing pressures greater than those units previously operated at the site</p>	<p>It is necessary to consider spudcan - footprint interaction issues as well as potential degradation of lateral capacity of jacket piles. Continuous piezocone tests from 0 to TD at each spudcan location with one continuous sampling borehole centrally located or at a spudcan location adjacent to a piezocone test, or suitable combination of continuous sampling and piezocone tests to determine the ground model^a</p> <p>or similar combinations of continuous sampling and piezocone tests to TD, increase or decrease workscope depending upon local geological variation^a</p>	
<p>Key ⊕ composite borehole (downhole piezocone tests and sampling) ● continuous sampling borehole ▲ continuous piezocone test</p>				
<p>NOTE 1 If appropriate, it is advisable that the jack-up geotechnical SI be conducted in consultation with the field development teams in order to optimize data acquisition.</p>				
<p>NOTE 2 Target depth (TD) is the greater of either 30 m or 1,5 times the spudcan diameters beneath the calculated spudcan tip penetration depth at the maximum preload.</p>				
<p>NOTE 3 At minimum, the ground model is generated to TD, with adequate data acquired for reliable definition of the model.</p>				
<p>^a The requirement for and specification of the geotechnical site investigation workscope should always be discussed and agreed with a suitable qualified and experienced offshore geotechnical engineer(s).</p>				

Annex E (informative)

Foundations — Additional information and alternative approaches

E.1 Guidance on A.9.3.2.2: — Penetration in clays — Bearing capacity factors of Housby and Martin

Presented below is the theoretical solution for the bearing capacity of circular conical foundations on clays of uniform and increasing strength with depth as provided by Housby and Martin^[A.9.3-5].

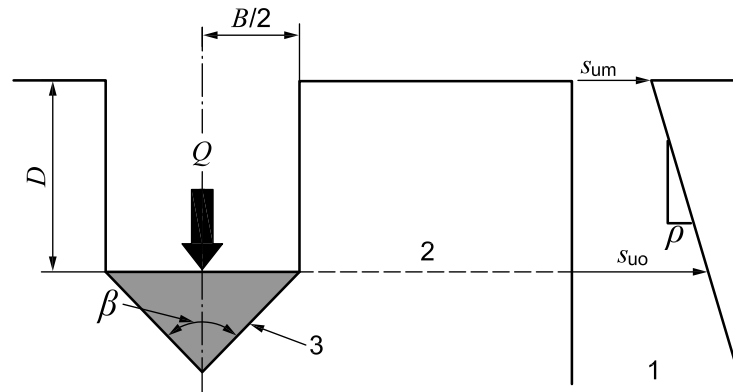
In Tables E.1-1 through E.1-5, the bearing capacity factors are defined for

- cone angles β of 60°, 90°, 120°, 150° and a flat plate of 180°;
- normalized 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 ρ is the rate of increase in undrained 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 mobilized 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. Tables E.1-1 through E.1-5 provide a theoretical lower bound to the bearing factor $N_c \cdot s_c \cdot d_c$ to apply to the shear strength at the spudcan base level, s_{uo} , for the full range of the above parameters, provided that D is not greater than H_{cav} as defined in A.9.3.2.1.4.

NOTE The bearing factor is nonlinear with respect to the non-dimensional soil strength gradient, embedment ratio and roughness factor. It is necessary to use caution when estimating appropriate bearing factors for non-dimensional soil strength gradients, embedment ratios and roughness factors other than those in the tables below.

**Key**

- 1 undrained shear strength profile
- 2 footing base level
- 3 footing/soil interface with adhesion (a_u) and roughness factor, α
- B effective spudcan diameter at uppermost part of bearing area
- D greatest depth of maximum cross-sectional spudcan bearing area
- Q bearing capacity at depth, D
- s_{um} undrained shear strength at sea floor
- s_{uo} undrained shear strength at footing base level: $s_{uo} = s_{um} + \rho D$
- a_u maximum shear stress that can be mobilized at the cone surface (adhesion)
- α roughness factor: $\alpha = a_u/s_u$
- β spudcan cone angle

NOTE Based on Tresca Yield Criterion for $\phi = 0$ and $\gamma = 0$.

Figure E.1-1 — Conical spudcan bearing capacity — Problem definition and notation

Table E.1-1 — Values of $N_{c.s.c.d_c} = [(Q_v/A) - p'_0]s_{u0}$ for cone angle $\beta = 60^\circ$

$\rho B/s_{um}$	D/B	Roughness factor $a = a_u/s_u$					
		0,0	0,2	0,4	0,6	0,8	1,0
0,0	0,0	4,45	4,96	5,45	5,90	6,32	6,69
0,0	0,1	4,68	5,19	5,67	6,12	6,53	6,90
0,0	0,25	4,98	5,50	5,96	6,40	6,81	7,18
0,0	0,5	5,41	5,90	6,37	6,81	7,21	7,57
0,0	1,0	6,07	6,55	7,01	7,43	7,84	8,18
0,0	2,5	7,33	7,81	8,25	8,66	9,05	9,39
1,0	0,0	5,81	6,51	7,15	7,77	8,34	8,87
1,0	0,1	5,92	6,59	7,23	7,83	8,38	8,89
1,0	0,25	6,04	6,70	7,30	7,88	8,42	8,91
1,0	0,5	6,20	6,84	7,41	7,96	8,47	8,94
1,0	1,0	6,43	7,05	7,58	8,12	8,59	9,03
1,0	2,5	6,97	7,55	8,08	8,54	8,98	9,39
2,0	0,0	7,14	8,02	8,84	9,60	10,32	10,99
2,0	0,1	6,92	7,73	8,49	9,21	9,88	10,50
2,0	0,25	6,74	7,50	8,18	8,84	9,46	10,03
2,0	0,5	6,59	7,29	7,91	8,53	9,09	9,61
2,0	1,0	6,55	7,20	7,76	8,33	8,83	9,30
2,0	2,5	6,99	7,49	8,03	8,50	8,95	9,37
3,0	0,0	8,49	9,54	10,50	11,42	12,29	13,10
3,0	0,1	7,77	8,70	9,56	10,38	11,14	11,85
3,0	0,25	7,24	8,03	8,80	9,53	10,20	10,82
3,0	0,5	6,82	7,56	8,21	8,86	9,45	10,00
3,0	1,0	6,60	7,27	7,85	8,44	8,94	9,43
3,0	2,5	6,99	7,47	8,01	8,49	8,94	9,36
4,0	0,0	9,83	11,02	12,16	13,24	14,26	15,18
4,0	0,1	8,51	9,52	10,48	11,38	12,22	13,00
4,0	0,25	7,61	8,44	9,26	10,04	10,75	11,41
4,0	0,5	6,97	7,74	8,41	9,08	9,69	10,26
4,0	1,0	6,64	7,31	7,90	8,49	9,01	9,51
4,0	2,5	6,86	7,45	8,00	8,48	8,94	9,35
5,0	0,0	11,17	12,52	13,83	15,06	16,20	17,26
5,0	0,1	9,14	10,23	11,26	12,25	13,15	13,99
5,0	0,25	7,90	8,78	9,63	10,43	11,17	11,87
5,0	0,5	7,08	7,84	8,55	9,24	9,86	10,45
5,0	1,0	6,66	7,32	7,94	8,53	9,06	9,56
5,0	2,5	6,85	7,44	7,99	8,47	8,93	9,35

Table E.1-2 — $N_{c.s.c.}d_c = [(Q_V/A) - p'_0]s_{uo}$ for cone angle $\beta = 90^\circ$

$\rho B/s_{um}$	D/B	Roughness factor $a = a_u/s_u$					
		0,0	0,2	0,4	0,6	0,8	1,0
0,0	0,0	4,64	5,02	5,36	5,67	5,95	6,17
0,0	0,1	4,90	5,28	5,61	5,91	6,18	6,41
0,0	0,25	5,22	5,59	5,93	6,23	6,49	6,71
0,0	0,5	5,68	6,03	6,36	6,66	6,92	7,14
0,0	1,0	6,37	6,71	7,05	7,32	7,58	7,79
0,0	2,5	7,65	8,03	8,32	8,60	8,86	9,05
1,0	0,0	5,57	6,05	6,47	6,87	7,22	7,53
1,0	0,1	5,74	6,21	6,62	7,00	7,36	7,65
1,0	0,25	5,94	6,38	6,79	7,16	7,50	7,79
1,0	0,5	6,16	6,61	6,99	7,36	7,68	7,97
1,0	1,0	6,50	6,93	7,30	7,64	7,95	8,21
1,0	2,5	7,25	7,57	7,94	8,25	8,53	8,78
2,0	0,0	6,46	7,03	7,54	8,01	8,45	8,82
2,0	0,1	6,41	6,94	7,43	7,88	8,28	8,65
2,0	0,25	6,41	6,88	7,35	7,76	8,14	8,46
2,0	0,5	6,40	6,88	7,29	7,69	8,03	8,35
2,0	1,0	6,54	6,99	7,37	7,73	8,06	8,33
2,0	2,5	7,16	7,49	7,86	8,18	8,47	8,72
3,0	0,0	7,36	8,00	8,59	9,14	9,65	10,08
3,0	0,1	6,99	7,57	8,10	8,60	9,05	9,45
3,0	0,25	6,70	7,24	7,73	8,17	8,59	8,94
3,0	0,5	6,54	7,04	7,47	7,88	8,24	8,57
3,0	1,0	6,56	7,02	7,41	7,78	8,11	8,39
3,0	2,5	7,12	7,46	7,83	8,15	8,44	8,46
4,0	0,0	8,22	8,96	9,64	10,25	10,82	11,33
4,0	0,1	7,49	8,11	8,68	9,22	9,70	10,14
4,0	0,25	6,94	7,50	8,01	8,48	8,92	9,29
4,0	0,5	6,63	7,15	7,58	8,01	8,38	8,72
4,0	1,0	6,57	7,03	7,43	7,80	8,14	8,42
4,0	2,5	7,05	7,44	7,81	8,13	8,42	8,67
5,0	0,0	9,11	9,93	10,66	11,35	12,00	12,56
5,0	0,1	7,87	8,55	9,17	9,74	10,26	10,75
5,0	0,25	7,12	7,71	8,24	8,72	9,17	9,57
5,0	0,5	6,70	7,22	7,67	8,09	8,47	8,82
5,0	1,0	6,57	7,04	7,44	7,82	8,16	8,44
5,0	2,5	7,03	7,42	7,80	8,12	8,41	8,66

Table E.1-3 — $N_{c,s_c,d_c} = [(Q_v/A) - p'_{o}] / s_{uo}$ for cone angle $\beta = 120^\circ$

$\rho B / s_{um}$	D/B	Roughness factor $a = a_u / s_u$					
		0,0	0,2	0,4	0,6	0,8	1,0
0,0	0,0	4,96	5,25	5,51	5,73	5,92	6,05
0,0	0,1	5,23	5,52	5,77	5,99	6,17	6,30
0,0	0,25	5,57	5,85	6,10	6,31	6,49	6,62
0,0	0,5	6,04	6,31	6,55	6,76	6,93	7,05
0,0	1,0	6,74	7,01	7,24	7,44	7,61	7,72
0,0	2,5	8,07	8,32	8,55	8,75	8,90	8,99
1,0	0,0	5,69	6,04	6,36	6,65	6,89	7,09
1,0	0,1	5,89	6,24	6,55	6,82	7,07	7,26
1,0	0,25	6,12	6,45	6,76	7,02	7,26	7,45
1,0	0,5	6,39	6,72	7,01	7,27	7,48	7,66
1,0	1,0	6,80	7,10	7,37	7,61	7,82	7,97
1,0	2,5	7,52	7,82	8,08	8,29	8,49	8,61
2,0	0,0	6,38	6,79	7,16	7,50	7,80	8,04
2,0	0,1	6,41	6,80	7,16	7,47	7,75	7,97
2,0	0,25	6,46	6,83	7,17	7,46	7,72	7,94
2,0	0,5	6,56	6,91	7,22	7,49	7,74	7,92
2,0	1,0	6,80	7,12	7,40	7,65	7,87	8,03
2,0	2,5	7,43	7,72	7,99	8,21	8,41	8,53
3,0	0,0	7,04	7,51	7,93	8,31	8,66	8,93
3,0	0,1	6,84	7,27	7,65	8,00	8,31	8,57
3,0	0,25	6,71	7,09	7,45	7,76	8,05	8,27
3,0	0,5	6,66	7,02	7,34	7,63	7,88	8,08
3,0	1,0	6,81	7,11	7,41	7,67	7,89	8,06
3,0	2,5	7,38	7,68	7,95	8,17	8,38	8,51
4,0	0,0	7,70	8,22	8,69	9,11	9,49	9,81
4,0	0,1	7,20	7,66	8,07	8,44	8,77	9,03
4,0	0,25	6,88	7,28	7,65	7,98	8,27	8,53
4,0	0,5	6,72	7,08	7,42	7,71	7,97	8,18
4,0	1,0	6,80	7,12	7,41	7,68	7,90	8,08
4,0	2,5	7,39	7,66	7,93	8,15	8,36	8,49
5,0	0,0	8,35	8,91	9,43	9,89	10,31	10,67
5,0	0,1	7,52	7,99	8,43	8,82	9,18	9,95
5,0	0,25	7,01	7,43	7,81	8,15	8,45	8,72
5,0	0,5	6,77	7,13	7,47	7,77	8,03	8,25
5,0	1,0	6,80	7,12	7,42	7,69	7,91	8,09
5,0	2,5	7,34	7,64	7,91	8,14	8,34	8,48

Table E.1-4 — $N_{c-s.c.d_c} = [(Q_V/A) - p'_0]/s_{u0}$ for cone angle $\beta = 150^\circ$

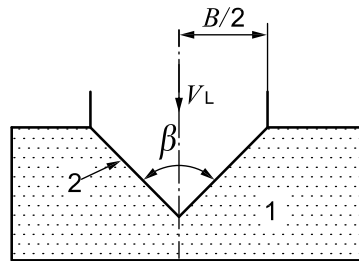
$\rho B/s_{um}$	D/B	Roughness factor $a = a_u/s_u$					
		0,0	0,2	0,4	0,6	0,8	1,0
0,0	0,0	5,32	5,55	5,74	5,89	6,01	6,05
0,0	0,1	5,60	5,82	6,00	6,16	6,26	6,30
0,0	0,25	5,94	6,16	6,34	6,49	6,59	6,61
0,0	0,5	6,41	6,62	6,80	6,94	7,03	7,05
0,0	1,0	7,13	7,32	7,49	7,62	7,71	7,72
0,0	2,5	8,46	8,65	8,81	8,93	8,99	8,99
1,0	0,0	5,94	6,22	6,46	6,67	6,84	6,97
1,0	0,1	6,16	6,43	6,67	6,87	7,04	7,15
1,0	0,25	6,41	6,67	6,90	7,09	7,25	7,36
1,0	0,5	6,71	6,96	7,18	7,36	7,51	7,60
1,0	1,0	7,13	7,36	7,57	7,73	7,86	7,95
1,0	2,5	7,91	8,12	8,31	8,44	8,56	8,61
2,0	0,0	6,50	6,82	7,11	7,35	7,57	7,73
2,0	0,1	6,59	6,90	7,16	7,40	7,59	7,74
2,0	0,25	6,69	6,98	7,23	7,45	7,63	7,76
2,0	0,5	6,84	7,10	7,34	7,54	7,70	7,82
2,0	1,0	7,11	7,35	7,57	7,74	7,89	7,99
2,0	2,5	7,81	8,01	8,21	8,35	8,47	8,53
3,0	0,0	7,03	7,40	7,72	7,98	8,24	8,43
3,0	0,1	6,94	7,27	7,56	7,81	8,03	8,21
3,0	0,25	6,88	7,18	7,45	7,68	7,88	8,03
3,0	0,5	6,91	7,18	7,43	7,63	7,81	7,94
3,0	1,0	7,10	7,35	7,57	7,75	7,90	8,00
3,0	2,5	7,76	7,97	8,16	8,31	8,43	8,49
4,0	0,0	7,55	7,94	8,30	8,58	8,88	9,10
4,0	0,1	7,23	7,58	7,89	8,16	8,40	8,59
4,0	0,25	7,02	7,34	7,62	7,86	8,07	8,23
4,0	0,5	6,95	7,23	7,49	7,70	7,88	8,01
4,0	1,0	7,09	7,34	7,56	7,75	7,90	8,00
4,0	2,5	7,72	7,94	8,13	8,29	8,41	8,47
5,0	0,0	8,05	8,48	8,86	9,19	9,48	9,74
5,0	0,1	7,46	7,83	8,16	8,44	8,69	8,90
5,0	0,25	7,13	7,45	7,74	7,99	8,20	8,37
5,0	0,5	6,99	7,27	7,53	7,74	7,93	8,07
5,0	1,0	7,09	7,34	7,56	7,75	7,91	8,01
5,0	2,5	7,70	7,93	8,12	8,27	8,40	8,46

Table E.1-5 — $N_{c-s.c.d_c} = [(Q_V/A) - p'_0]s_{u0}$ for cone angle $\beta = 180^\circ$

$\rho B/s_{um}$	D/B	Roughness factor $a = a_u/s_u$					
		0,0	0,2	0,4	0,6	0,8	1,0
0,0	0,0	5,69	5,86	5,97	6,03	6,05	6,05
0,0	0,1	5,97	6,13	6,24	6,29	6,30	6,30
0,0	0,25	6,31	6,47	6,57	6,61	6,61	6,61
0,0	0,5	6,79	6,93	7,02	7,05	7,05	7,05
0,0	1,0	7,49	7,63	7,70	7,71	7,71	7,71
0,0	2,5	8,82	8,94	8,99	8,99	8,99	8,99
1,0	0,0	6,25	6,47	6,65	6,79	6,90	6,95
1,0	0,1	6,48	6,69	6,87	7,00	7,10	7,14
1,0	0,25	6,74	6,94	7,11	7,23	7,32	7,35
1,0	0,5	7,05	7,24	7,39	7,51	7,58	7,60
1,0	1,0	7,47	7,64	7,79	7,88	7,93	7,94
1,0	2,5	8,26	8,32	8,52	8,60	8,61	8,61
2,0	0,0	6,73	6,98	7,20	7,39	7,53	7,63
2,0	0,1	6,85	7,08	7,30	7,46	7,59	7,68
2,0	0,25	6,98	7,20	7,39	7,55	7,66	7,72
2,0	0,5	7,15	7,36	7,53	7,67	7,76	7,80
2,0	1,0	7,45	7,63	7,78	7,90	7,96	7,98
2,0	2,5	8,16	8,27	8,43	8,50	8,53	8,53
3,0	0,0	7,16	7,45	7,69	7,91	8,08	8,21
3,0	0,1	7,13	7,40	7,62	7,81	7,96	8,07
3,0	0,25	7,15	7,37	7,58	7,75	7,88	7,96
3,0	0,5	7,21	7,42	7,61	7,75	7,86	7,91
3,0	1,0	7,43	7,62	7,78	7,90	7,97	7,99
3,0	2,5	8,13	8,23	8,38	8,46	8,49	8,49
4,0	0,0	7,56	7,87	8,15	8,38	8,58	8,73
4,0	0,1	7,38	7,64	7,89	8,09	8,26	8,39
4,0	0,25	7,26	7,50	7,71	7,89	8,03	8,13
4,0	0,5	7,25	7,46	7,65	7,80	7,92	7,98
4,0	1,0	7,44	7,61	7,77	7,89	7,97	8,00
4,0	2,5	8,09	8,19	8,36	8,44	8,47	8,47
5,0	0,0	7,94	8,27	8,57	8,83	9,05	9,23
5,0	0,1	7,56	7,85	8,10	8,32	8,50	8,64
5,0	0,25	7,34	7,59	7,81	8,00	8,15	8,25
5,0	0,5	7,27	7,49	7,68	7,84	7,96	8,02
5,0	1,0	7,43	7,60	7,77	7,89	7,97	8,00
5,0	2,5	8,07	8,18	8,35	8,43	8,46	8,46

E.2 Guidance on A.9.3.2.4 — Penetration in silica sands

Theoretical values of N_γ calculated by Cassidy and Houlsby^[E.2-1] using the slip-line method for circular footings with a conical underside are given in Tables E.2-1 to E.2-5 (see also Figure E.2-1). These values cover cone apex angles from 60° to 180°, spudcan-soil interface roughness coefficients ($\alpha = \tan \delta / \tan \phi'$) from 0,6 to 1, and soil friction angles from 20° to 40°. It is noted that the N_γ values are non-linear with ϕ' and this should be accounted for in any interpolation of Tables E.2-1 to E.2-5. The soil-steel interface friction angle, δ , is typically in the range 22° to 29°, decreasing with increasing grain size AP^[E.2-2]. This implies that the roughness coefficient, α , is at least 0,6 for typical soil friction angles ($\phi' = 30^\circ \pm 5^\circ$). Be aware 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 $\alpha = 1$) were calculated using a different implementation of the slip-line method, Martin^[A.9.3-8].



Key

- 1 homogeneous soil
- 2 spudcan-soil interface, with roughness coefficient α
- $B/2$ effective spudcan radius
- β spudcan cone apex angle
- V_L available spudcan reaction; see Equation (A.9.3-1)

Figure E.2-1 — Definition of parameters for Tables E.2-1 to E.2-5

Table E.2-1 — Bearing capacity factors (N_γ) for a conical apex angle of 60°

$\beta = 60^\circ$			
$\phi' (^\circ)$	$\alpha = 1$	$\alpha = 0,8$	$\alpha = 0,6$
20	7,33	6,55	5,64
25	14,69	12,99	10,94
30	31,99	27,45	22,50
35	79,26	62,95	48,81
40	209,20	163,20	122,30

Table E.2-2 — Bearing capacity factors (N_γ) for a conical apex angle of 90°

$\beta = 90^\circ$			
$\phi' (^\circ)$	$\alpha = 1$	$\alpha = 0,8$	$\alpha = 0,6$
20	4,54	4,11	3,62
25	9,58	8,50	7,26
30	21,12	18,87	15,58
35	51,76	47,42	37,01
40	142,80	132,60	99,18

Table E.2-3 — Bearing capacity factors (N_γ) for a conical apex angle of 120°

$\beta = 120^\circ$			
$\phi' \text{ (}^\circ\text{)}$	$\alpha = 1$	$\alpha = 0,8$	$\alpha = 0,6$
20	3,37	3,15	2,81
25	7,46	6,99	6,05
30	17,58	16,80	14,30
35	44,73	42,99	35,79
40	129,40	124,90	103,30

Table E.2-4 — Bearing capacity factors (N_γ) for a conical apex angle of 150°

$\beta = 150^\circ$			
$\phi' \text{ (}^\circ\text{)}$	$\alpha = 1$	$\alpha = 0,8$	$\alpha = 0,6$
20	2,72	2,61	2,39
25	6,44	6,13	5,60
30	15,93	15,13	14,02
35	42,36	40,42	36,41
40	128,10	120,50	110,50

Table E.2-5 — Bearing capacity factors (N_γ) for a conical apex angle of 180°

$\beta = 180^\circ$			
$\phi' \text{ (}^\circ\text{)}$	$\alpha = 1$	$\alpha = 0,8$	$\alpha = 0,6$
20	2,16	2,04	1,99
25	5,27	5,37	5,14
30	14,13	13,91	12,98
35	42,56	40,93	36,81
40	129,40	121,50	117,00

The bearing capacity calculated using Equation (A.9.3-8) is strongly dependent on the adopted soil friction angle, ϕ' . The apparent friction angle mobilized during spudcan penetration in sand is influenced by

- a) the soil relative density (and therefore the dilatancy): peak friction angle increases with relative density;
- b) the size of the spudcan, and, therefore, the stress level within the failing soil: peak friction angle decreases as the stress level increases;
- c) progressive failure: soil elements at different locations within the failure mechanism have undergone widely differing levels of shear strain;
- d) progressive failure due to pre-shearing of the soil by the conical spudcan tip, which acts to reduce the mobilized peak strength;
- e) compression of the foundation soil, which generates additional settlement;
- f) 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 a) and b) 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 ~10 % of the bearing pressure is typically found to be appropriate (see, e.g. Perkins and Madson^[E.2-3], Randolph *et al.*^[E.2-4] and White *et al.*^[E.2-5]). Alternatively, correlations with CPT parameters can be used to assess the spudcan penetration directly according to Schmertmann *et al.*^[E.2-6] and Schmertmann^{[E.2-7][E.2-8]}, or to infer the soil relative density, from which the peak friction angle can be estimated.

However, the apparent friction angle mobilized during spudcan penetration is lower than the peak value measured in the laboratory (or inferred using CPT correlations), due to mechanisms c) to e) above. Back-analyses of field penetration records (Cassidy *et al.*^[E.2-9]) and centrifuge tests (White *et al.*^[E.2-5]) have indicated that the friction angle is similar to the critical state friction angle, increasing by up to 5° with increasing relative density.

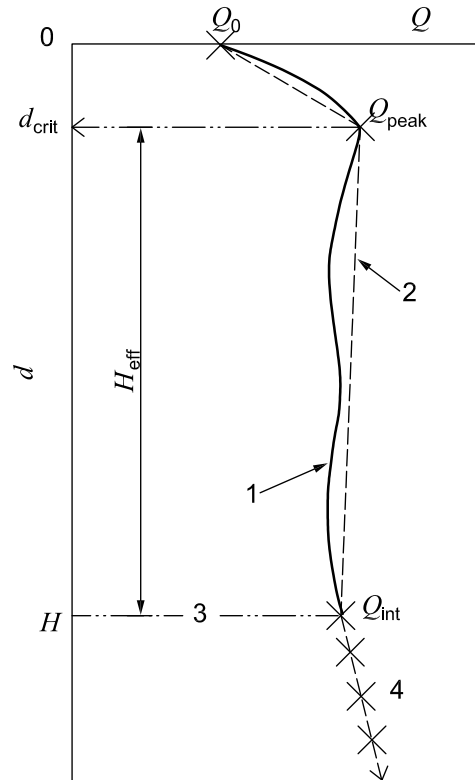
If preloading is conducted too quickly for drained conditions to prevail, then positive excess pore pressures can be generated beneath the spudcan, leading to a reduction in bearing capacity [mechanism f) above]. This possibility is particularly relevant for skirted spudcans.

E.3 Guidance on A.9.3.2.6.4 — Punch-through — Sand overlying clay — Further details on alternate methods

Recent research reported by Hossain *et al.*^[E.3-1], Lee *et al.*^[E.3-2] and Teh *et al.*^[E.3-3] has proposed alternative methodologies to those described in A.9.3.2.6.3 and A.9.3.2.6.4 for assessing the bearing capacity of spudcans in layered soil conditions. These are based on the results of centrifuge model experiments and include an evaluation of the penetration resistance of the soil plug formed by the punch-through failure mechanism into the underlying clay.

The specific details of each method are described in Lee^[E.3-4], Teh *et al.*^[E.3-5] and Teh^[E.3-6]; a précis of the method of Teh *et al.*^[E.3-5] for analysing the situation where sand overlies extensive clay is included in the following sections.

Teh *et al.*^[E.3-5] 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 ($Q_{\text{nom}} - d$) profile based on a simplified profile shown in Figure E.3-1. The spudcan bearing resistance can hence be represented by the bearing capacity at sea floor (i.e. $d = 0$), Q_0 , followed by the maximum bearing capacity, Q_{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_{crit} refers to the depth where punch-through occurs and H is the thickness of the upper sand layer.



Key

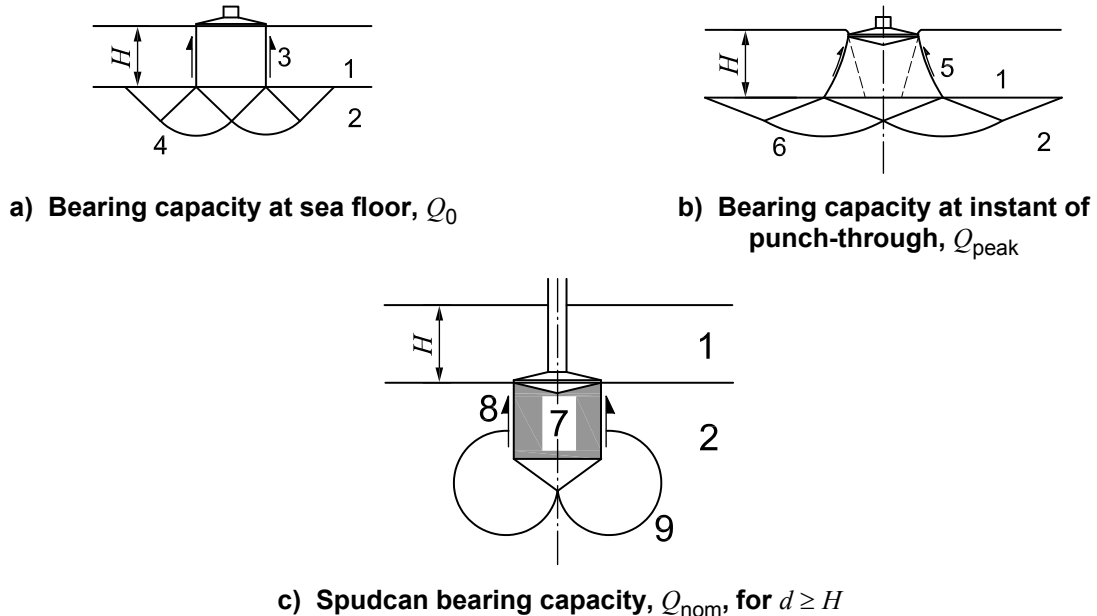
- 1 measured spudcan bearing capacity
- 2 simplified interpretation of measured spudcan bearing capacity
- 3 sand-clay interface
- 4 nominal spudcan bearing capacity in clay = Q_{nom} for $d \geq H$
- H thickness of upper sand layer
- H_{eff} effective thickness of upper sand layer
- Q_0 spudcan bearing capacity at sea floor
- Q_{peak} peak spudcan bearing capacity in sand layer at the critical depth
- Q_{int} spudcan bearing capacity at the sand-clay interface
- Q spudcan bearing capacity
- d depth, expressed in metres
- d_{crit} critical depth where punch-through occurs

Figure E.3-1 — Measured and simplified spudcan ultimate bearing capacity-depth profile in sand overlying clay

The formulations for estimating the three characteristic ultimate bearing capacities are given in Teh *et al.*^[E.3-3] and summarized 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.*^[E.3-7], Burd and Frydman^[E.3-8] and Teh *et al.*^[E.3-5]). 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 and Chua^[E.3-9]) 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.

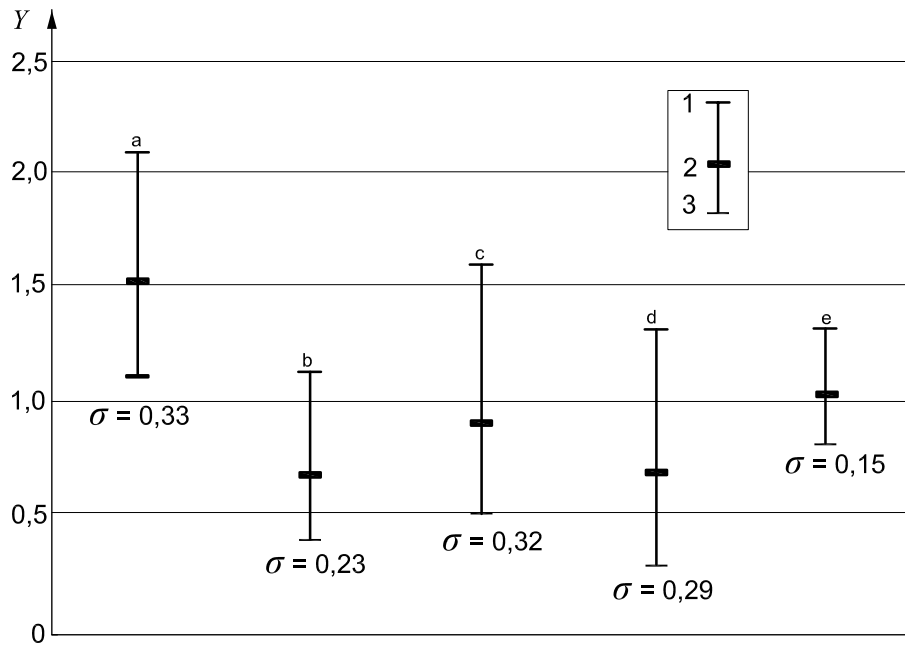


Key

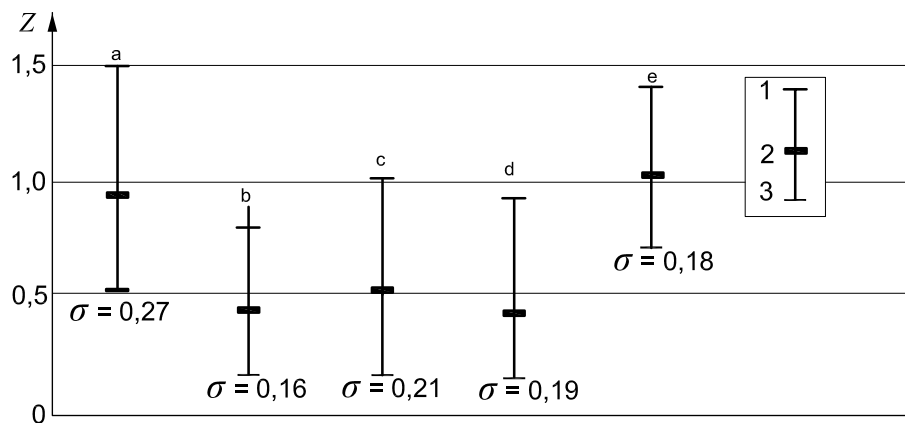
- 1 sand layer
- 2 clay layer
- 3 shearing along a vertical plane
- 4 general bearing shear failure
- 5 shearing along logarithmic spiral failure surface
- 6 clay bearing shear failure subjected to vertical and inclined loads
- 7 sand plug
- 8 side friction
- 9 deep flow mechanism in clay
- H thickness of sand layer
- Q_0 bearing capacity at sea floor
- Q_{peak} bearing capacity at instant of punch-through
- Q_{nom} spudcan bearing capacity when $d \geq H$
- d depth of spudcan below top of sand layer

Figure E.3-2 — Schematic of spudcan failure mechanisms

Figures E.3-3 a) and E.3-3 b) compare the ratio of calculated over measured Q_0 and Q_{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.*^[E.3-10] and Craig and Chua^[E.3-9]. The above verification is based on limited test data and further studies can be necessary to evaluate the above proposed approach in the determination of spudcan punch-through in sand overlying clay.



a) Summary of $Q_{0,calculated}/Q_{0,measured}$ ratio given by various methods



b) Summary of $Q_{peak,calculated}/Q_{peak,measured}$ ratio given by various methods

Key

- 1 maximum value
- 2 mean value
- 3 minimum value

Y ratio of calculated bearing capacity over measured bearing capacity ($Q_{0,calculated}/Q_{0,measured}$) based on various calculation methods (as described for lines designated as ^a to ^e) and 18 centrifuge tests reported by Teh *et al.*^[E.3-10]

Z ratio of calculated peak bearing capacity over measured peak bearing capacity ($q_{peak,calculated}/q_{peak,measured}$) based on various calculation methods (as described for lines designated as ^a to ^e) and 18 centrifuge tests reported by Teh *et al.*^[E.3-10]

σ standard deviation

^a punching shear (Hanna and Meyerhof^[A.9.3-32])

^b punching shear (SNAME^[7])

^c projected area – 1:3 (SNAME^[7])

^d projected area – 1:5 (SNAME^[7])

^e Teh *et al.*^[E.3-3]

Figure E.3-3 — Comparison calculated and experimental bearing capacities

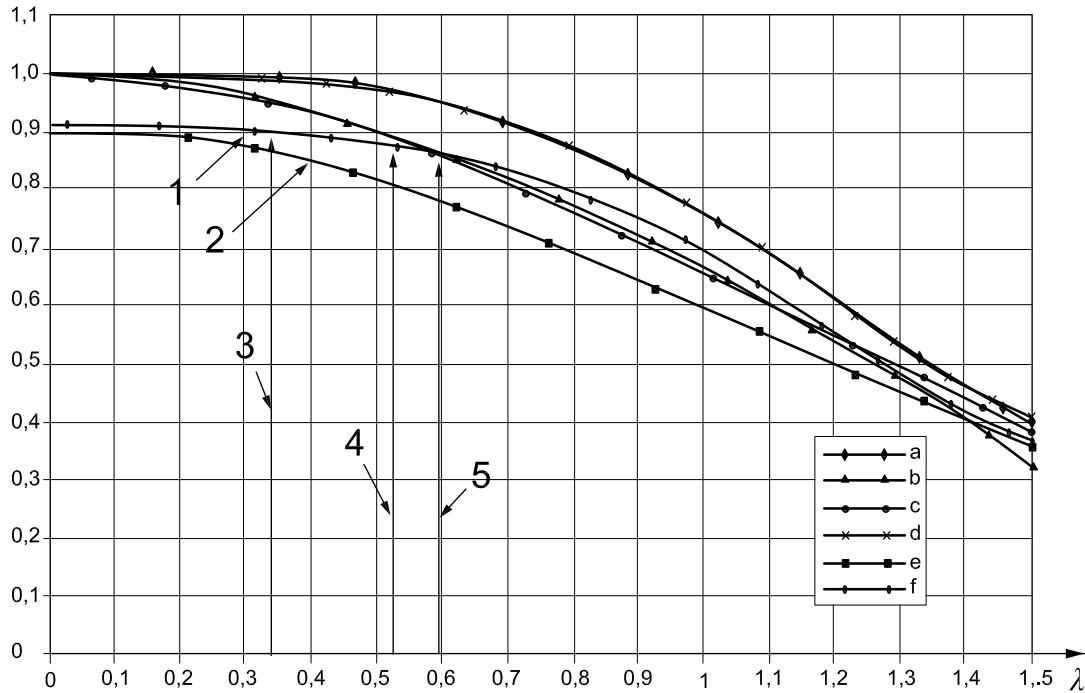
Annex F (informative)

Informative annex on Clause A.12 — Structural strength

F.1 Guidance on A.12.6.2.4 — Axial compressive column buckling strength

The formulations in AISC 2005^[A.12.5-1] are based on the SSRC column buckling curves^[A.12.6-1] primarily curve 2P. Prior to the introduction of AISC 2005^[A.12.5-1], the compression curve addressed fabricated sections from mill run steel and different material standards covering a range of material yield. In the development of AISC 2005, the reliability evaluation was updated to reflect the general use of 345 N/mm² material, better steel manufacturing practices, and the limited use of mill steel in fabrication. This review supported the increase in the partial resistance factor for compression to 0,9 (equivalent to 1/0,9 in this part of ISO 19905).

The referenced AISC column curve represents curve 2P for λ up to Euler Buckling. Curve 2P, and thus the AISC curve, relates primarily to traditional building construction steels typically with yield stresses up to 345 N/mm², open sections such as wide flanges, and non-symmetrical sections together with their corresponding column tolerances of length/1 000. The AISC equation also applies to prismatic members fabricated from production runs of steel plates and rolled sections. When this type of prismatic member is used on a jack-up the traditional column equations should be used. However, SSRC found that for high strength steel, up to 700 N/mm², as used in the construction of chords, and for closed sections the SSRC curve 1P is applicable. The high-strength steels used in jack-up fabrication are required to meet higher manufacturing requirements (e.g. limits on charpy impact, chemistry and improved welding procedures). In addition, the tighter column tolerances at around length/1 500 for jack-up chord fabrication results in better geometry control than a production run of rolled sections. The effects of residual stresses due to welding and rolling reduce as the yield stress of the material increases. For these reason, the “high-strength” SSRC column curve 1P is appropriate for the assessment of chord members. A good approximation to this, within 0,8 % of the SSRC expression from $\lambda = 0,0$ to 2,0, is shown in Figure F.1-1 and used in Equations (A.12.6-18) and (A.12.6-19) in A.12.6.2.4 b).



Key

- 1 allowable curve for high strength steel ($F_y > 450 \text{ N/mm}^2$)
- 2 allowable curve for normal strength steel ($F_y < 450 \text{ N/mm}^2$)
- 3 λ for “tear drop” shaped chord with cross-section area of $0,091 \text{ m}^2$ and $3,35 \text{ m}$ bay height
- 4 λ for “tear drop” shaped chord with cross-section area of $0,158 \text{ m}^2$ and $4,88 \text{ m}$ bay height
- 5 λ for a split tubular chord with cross-section area of $0,079 \text{ m}^2$ and $3,35 \text{ m}$ bay height
- λ column slenderness parameter as defined in A.12.6.2.4
- $P_p/(AF_{y\text{eff}})$ ratio of maximum allowable axial force based on member slenderness to axial force required to yield entire cross-section
- a SSRC column curve 1P.
- b SSRC column curve 2P.
- c ISO 19905-1 curve for normal strength steel ($F_y < 460 \text{ N/mm}^2$).
- d ISO 19905-1 curve for high strength steel ($F_y > 460 \text{ N/mm}^2$).
- e Allowable curve for normal steel (including the partial resistance factor of 1,1).
- f Allowable curve for high strength steel (including the partial resistance factor of 1,1).

Figure F.1-1 — Comparison of column curves

F.2 Guidance on A.12.6.3.2 — Interaction equation approach — Determination of η

Determination of the correct value of η 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 does 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 is greatly affected.

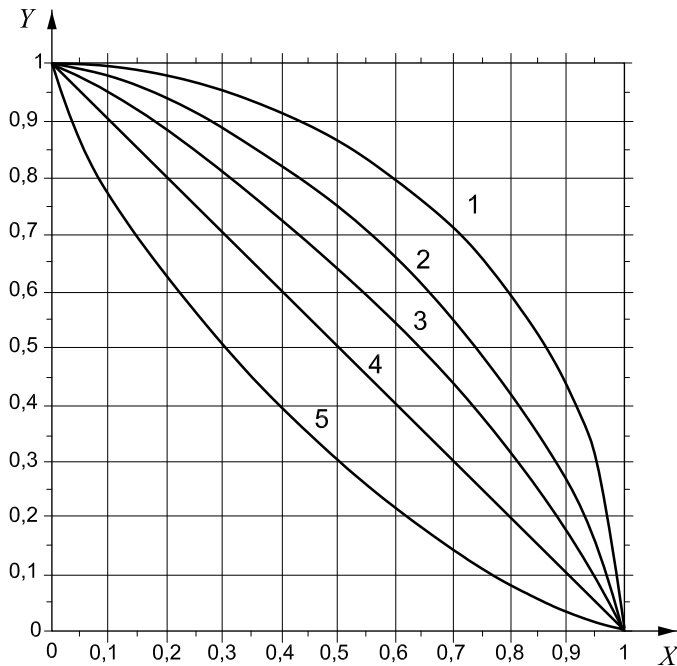
Once the nominal bending strength has been calculated for a few angles between the x- and y-axes, the value for η can be calculated using a graphical procedure (see Figure F.2-1), or by an iterative procedure. It has been found that a successful iterative procedure is the use of coupled equations, setting $a = M'_{uey}/M_{nx}$ and $b = M'_{uey}/M_{ny}$ resulting in Equation (F.2-1):

$$\eta_{i+1} = \frac{\ln(1 - b^\eta)}{\ln(a)} \tag{F.2-1}$$

where the initial value $\eta = 1,5$ and the accelerating step is as given by Equation (F.2-2):

$$\eta_{i+2} = 0,5(\eta_{i+1} + \eta_i) \tag{F.2-2}$$

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 η . This can still make the equation conservative although considerably less so than for $\eta = 1,0$.



The interaction equation exponent for biaxial bending, η , is calculated from the equation

$$\left[\left(\frac{M_{uaz}}{M_{buz}} \right)^\eta + \left(\frac{M_{uay}}{M_{by}} \right)^\eta \right]^{\frac{1}{\eta}} = 1,0$$

Key

- X ratio of the applied bending moment to the allowable bending moment about the z axis: M_{uaz}/M_{buz}
- Y ratio of the applied bending moment to the allowable bending moment about the y axis: M_{uay}/M_{by}
- 1 interaction equation exponent for biaxial bending, $\eta = 2,0$
- 2 interaction equation exponent for biaxial bending, $\eta = 1,5$
- 3 interaction equation exponent for biaxial bending, $\eta = 1,25$
- 4 interaction equation exponent for biaxial bending, $\eta = 1,0$
- 5 interaction equation exponent for biaxial bending, $\eta = 0,75$

Figure F.2-1 — Graphical approach to the determination of η

F.3 Guidance on A.12.6.3.3 — Interaction surface approach

The following interaction surfaces and data are based on Dyer^[F.2-1]. There are likely to be other sections that are not included in the tables associated with each diagram. Sections not included in F.3 should be developed from first principles, see for example Duan and Chen^[F.2-2]. For sections that are unsymmetric about a particular axis, the proper positive sign conventions for the moments are given on the cross-sectional diagrams.

The utilization developed for each cross section is calculated for a specific value of P/P_y . The limiting utilization is based on a function of the two applied moments, M_y and M_z , and the two moment capacities, M'_{py} and M'_{pz} , for a specific value of P/P_y . It takes account of the amount of axial load only by specifying which of the concentric plots should be used. It is, therefore, possible to have a high axial utilization (i.e. a high value of P/P_y) but a low overall member utilization because there are small moments M_y and M_z . While this gives the

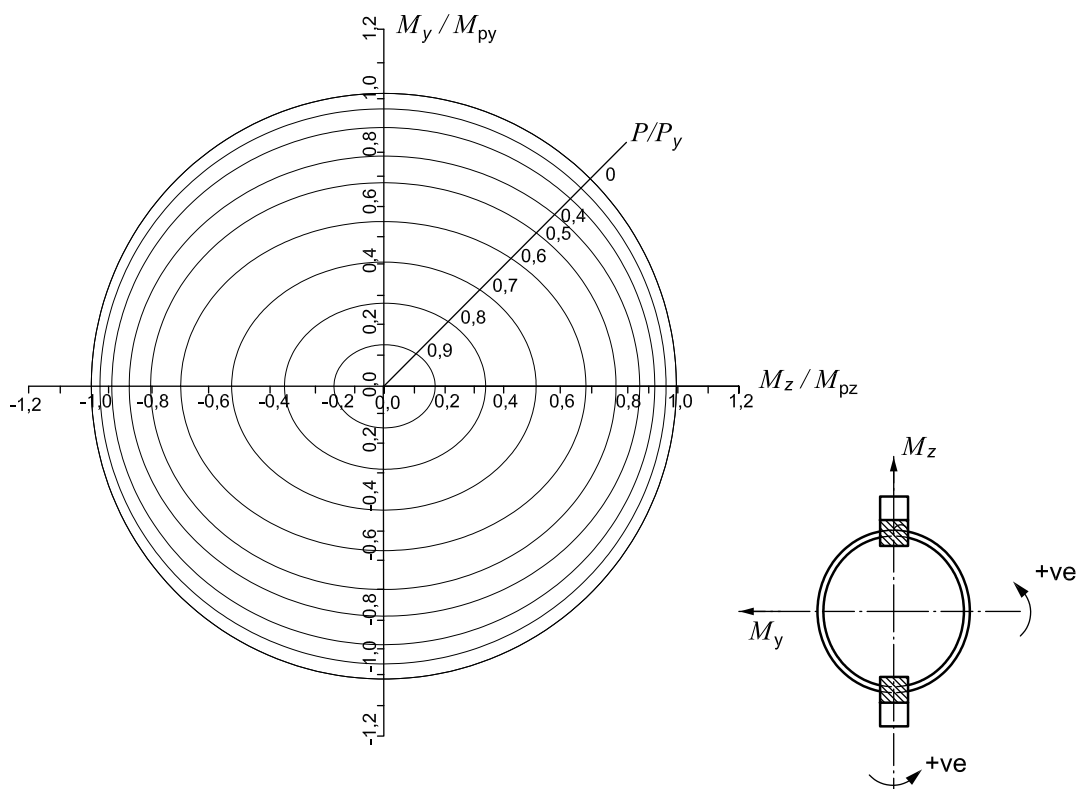
correct limiting utilization, it can give an unrealistic impression of the member utilization when compared to unity. In order to correct this, a relative utilization $U_{\text{int.rel}}$ can be approximated by determining the function as given by Equation (F.3-1):

$$U_{\text{int.rel}} = \left(\frac{P}{P_y} \right) + \left(1 - \frac{P}{P_y} \right) \left[\left(\frac{M_y}{M'_{py}} \right)^\xi + \left(\frac{M_z / M_{pz} - K}{M'_{pz} / M_{pz} - K} \right)^\xi \right]^{1/\xi} \quad (\text{F.3-1})$$

where the parameters are defined in Figures F.3-1 to F.3-4 and K is zero except when assessing triangular chords with single racks (Figure F.3-4). It is important that $U_{\text{int.rel}}$ not be used for any purpose except to give a measure of the overall section relative utilization; the check against unity is given in each of the individual Figures F.3-1 to F.3-4.

For the strength check, the applied member forces (P, M_y, M_z in Annex F) should be P_u, M_{uey}, M_{uez} as defined in A.12.6.3.2.

For the beam-column check, the applied member forces (P, M_y, M_z in Annex F) should be P_u, M_{uay}, M_{uaz} as defined in A.12.6.3.2.



Strength interaction equations (for mapping of the variables; see A.12.6.3.3) are as given by Equations (F.3-2):

$$\left\{ \left(\frac{M_y}{M'_{py}} \right)^2 + \left(\frac{M_z}{M'_{pz}} \right)^2 \right\}^{1/2} \leq 1,0 \quad (\text{F.3-2})$$

where

$$\text{for } (P/P_y) \leq 0,6: \quad M'_{py} = M_{py} \left[\cos \left(\frac{\pi P}{2P_y} \right) \right]^{1,1}$$

$$M'_{pz} = M_{pz} \left[\cos \left(\frac{\pi P}{2P_y} \right) \right]^{0,7}$$

$$\text{for } 1,0 > (P/P_y) > 0,6: \quad M'_{py} = 1,39M_{py} \left(1 - \frac{P}{P_y} \right)$$

$$M'_{pz} = 1,71M_{pz} \left(1 - \frac{P}{P_y} \right)$$

When $(P/P_y) \geq 1,0$, the member has failed.

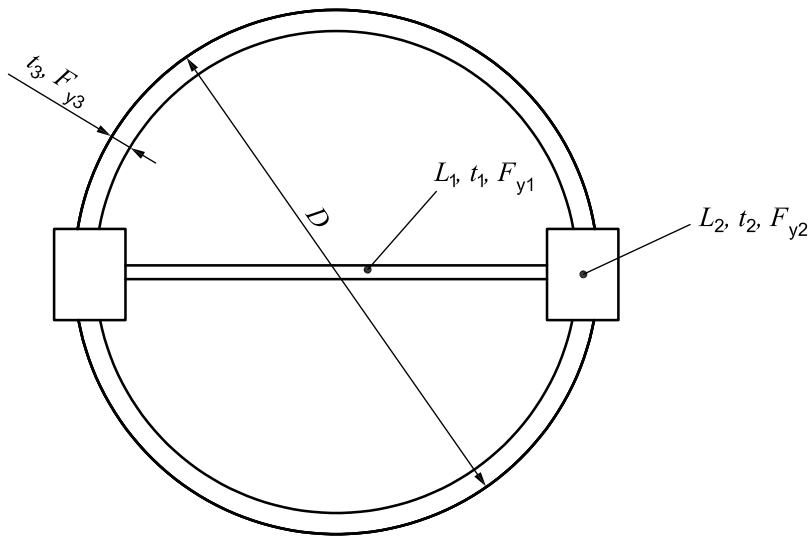
Key

P	chord member axial force
P_y	chord member axial strength
M_y and M_z	local y- and z-axis bending moments
M_{py} and M_{pz}	local y- and z-axis bending strengths
M'_{py} and M'_{pz}	adjusted local y- and z-axis bending strengths used in simplified interaction equations

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:



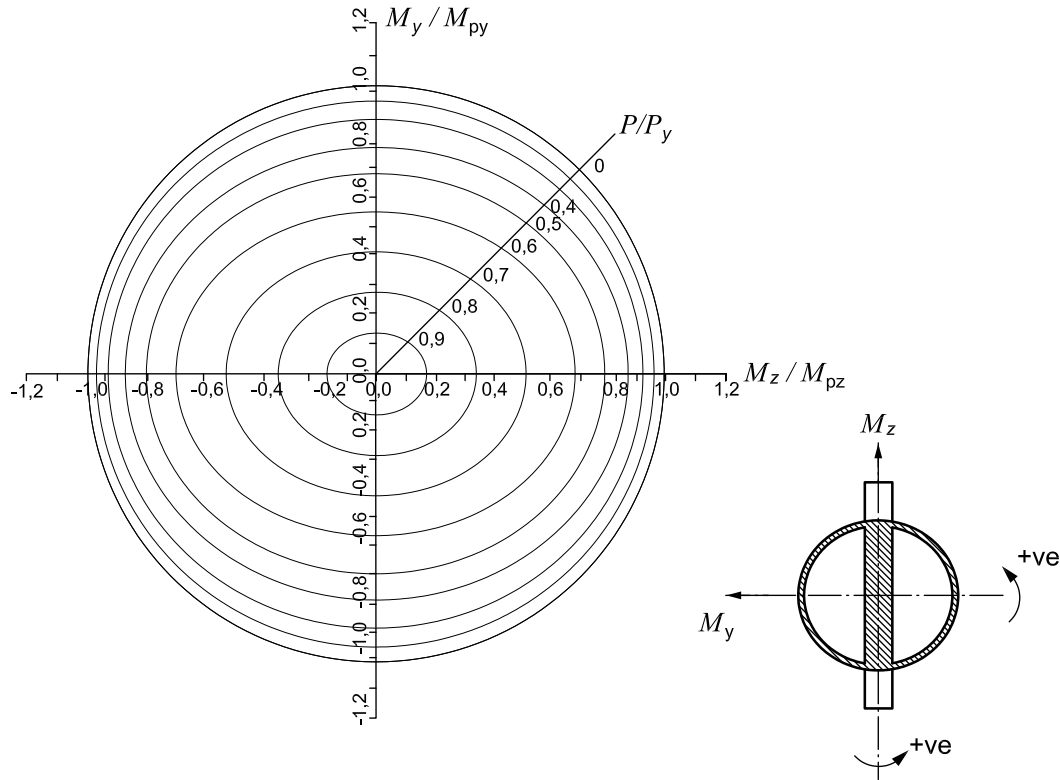
All dimensions are in millimetres, yield stresses are in MPa

Design	L_1	t_1	L_2	t_2	D	t_3	<--- Yield ---> Stress			Bay Ht			
							F_{y1}	F_{y2}	F_{y3}				
BMC JU-300-CAN (Zapata Scotian)	991	127	0	0	914	44	690	0	690	5532			
CFEM T2001 (Hitachi Redesign)	960	18	121	140	960	52	690	690	690	4500 Btm 3 bays 4100 Top 3 bays 4050 Middle bays			
						34							
						26							
						34							
						42							
CFEM T2005	650	20	108	140	800	28	700	685	650 700*	or 5050			
						30							
						31							
						32							
						33							
						35							
						36							
						38							
						40					700	685	700
						44					700	685	650
34	700	685	650										
38													
42													

* Early CFEM T2005 designs use 650 MPa steel for tube; later designs use 700 MPa steel.

Table F.3-1 (continued)

Design	L_1	t_1	L_2	t_2	D	t_3	<--- Yield ---> Stress			Bay Ht
							F_{Y1}	F_{Y2}	F_{Y3}	
CFEM T2600	650	20	120	140	800	33	700	700	700	6000
						35				
						38				
						40				
						41				
						43				
						45				
						47				
						49				
						50				
						51				
						52				
						55				
						56				
						57				
MODEC 200	450	15	102	127	559	27	490	690	490	5486
		27								
MODEC 300	450	25	102	127	559	34	490	690	490	5486
		28								
		15								
		20								
		27								
		60								
MODEC 400 (Trident 9)	690	115	102	127	800	40	490	690	490	6200
		20								
		20								
		35								
		35								
Hitachi K1025/31/32	900	18	100	127	900	32	690	690	690	5160
		18								
		18								
		20								
Hitachi K1026 (Neddrill 4)	950	20	100	127	950	42	690	690	690	4360
		18								
		18								
		20								
Hitachi K1056/7	1000	28	130	178	1000	47	690	730	690	4600
		30								
		30								
		30								
		30								
		60								
ETA Robray 300 (Asia Class)	627	60	127	127	762	64	690	690	690	4000
		60								
		11								
		13								
		14								
		16								
		17								
		19								
		25								
		32								
ETA Europe Class	627	38	140	140	762	22	690	690	690	5486
		51								
		64								
		76								
		89								
		102								
		114								
		127								



Strength interaction equations (for mapping of the variables; see A.12.6.3.3) are as given by Equations (F.3-3):

$$\left[\left(\frac{M_y}{M'_{py}} \right)^2 + \left(\frac{M_z}{M'_{pz}} \right)^2 \right]^{1/2} \leq 1,0 \tag{F.3-3}$$

where

$$\text{for } (P/P_y) < 1,0: \quad M'_{py} = M_{py} \left[1 - \left(\frac{P}{P_y} \right)^{1,85} \right]$$

$$\text{for } (P/P_y) < 1,0: \quad M'_{pz} = M_{pz} \left[1 - \left(\frac{P}{P_y} \right)^{2,25} \right]$$

When $(P/P_y) \geq 1,0$ the member has failed.

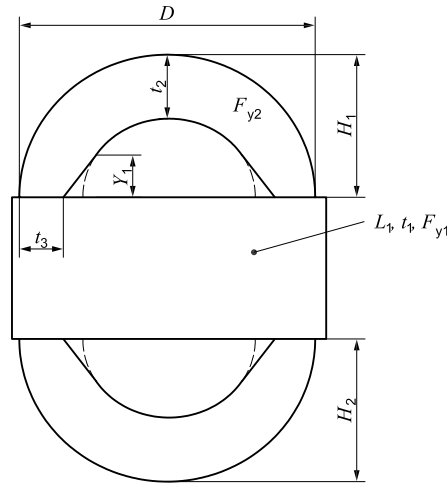
Key

- P chord member axial force
- P_y chord member axial strength
- M_y and M_z local y- and z-axis bending moments
- M_{py} and M_{pz} local y- and z-axis bending strengths
- M'_{py} and M'_{pz} adjusted local y- and z-axis bending strengths used in simplified interaction equations

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



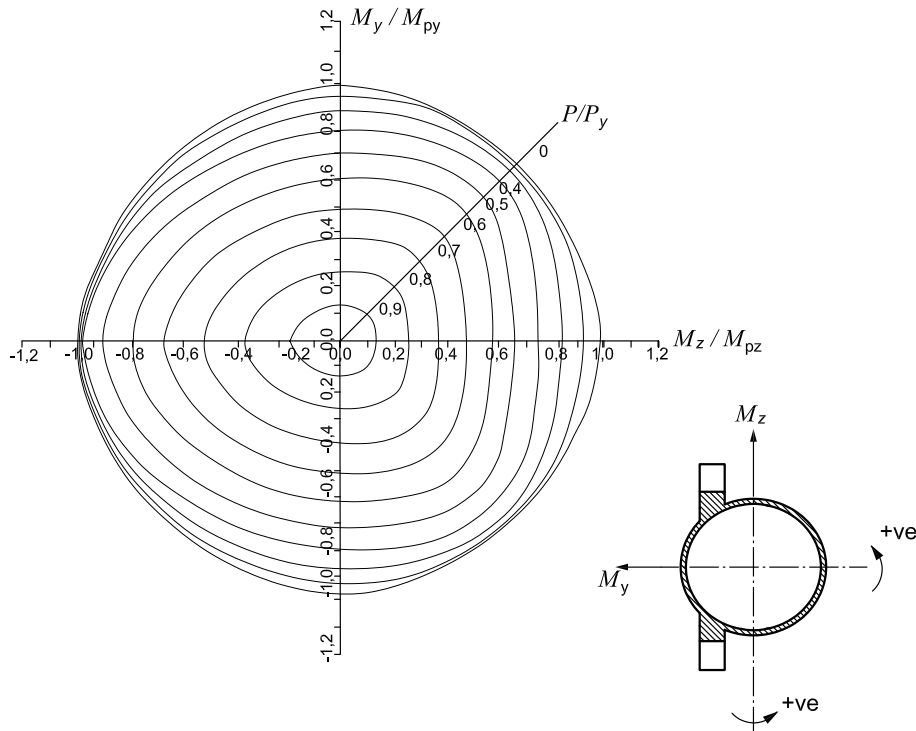
All dimensions are in millimetres, yield stresses are in MPa

a) Sections used in the derivation of Figure F.3-2 and Equation (F.3-3):

Design	Yield stress													Bay Ht
	L_1	t_1	D	t_2	t_3	L_4	t_4	Y_1	H_1	H_2	F_{y1}	F_{y2}		
F & G L780 (Lower bays)	400	152	381	25	25	0	0	0	191	165	621	690	3658	
F & G L780 (Upper bays)	400	127	381	25	25	0	0	0	191	191	621	450	3658	
F & G L780 m2 (Lower bays)	400	152	381	32	32	0	0	0	191	165	621	690	3658	
F & G L780 m2 (Upper bays)	400	127	381	32	32	0	0	0	191	191	621	517	3658	
F & G L780 m5 (Monitor)	401	178	381	81	57	0	0	51	178	178	690	690	4267	
F & G L780 m5 (Monarch)	401	178	381	81	51	0	0	51	178	178	690	690	4267	
F & G L780 m6	611	178	584	83	38	0	0	95	292	292	690	690	5486	
MSC CJ62 (Lower bays)	650	210	600	65	48	0	0	75	270	270	690	690	6927	
MSC CJ62 (Upper bays)	650	210	600	55	40	0	0	75	270	270	690	690	6927	
MSC CJ50 (1) (Concept)	550	210	520	25	25	0	0	0	260	260	690	690	5608	
MSC CJ50 (2) (Concept)	550	210	520	25	35	0	0	0	260	260	690	690	5608	
Technip TPG 500 (1)	722	160	680	75	61	0	0	20	340	340	690	540	6000	
Technip TPG 500 (2)	722	160	680	75	37	0	0	55	340	340	690	540	6000	
Technip TPG 500 (3)	722	160	680	62	37	0	0	36	340	340	690	540	6000	
Technip TPG 500 (4)	722	160	680	58	37	0	0	30	340	340	690	540	6000	
Technip TPG 500 (5)	722	160	680	50	37	0	0	19	340	340	690	540	6000	
Technip TPG 500 (6)	722	160	680	50	37	510	30	19	340	340	690	540	6000	

b) Additional data for newer sections that were NOT used in the derivation of Figure F.3-2 and Equation (F.3-3). Figure F.3-2 and Equation (F.3-3) should not be used for their evaluation:

Design	Yield stress													Bay Ht
	L_1	t_1	D	t_2	t_3	L_4	t_4	Y_1	H_1	H_2	F_{y1}	F_{y2}		
LeTourneau TARZAN	381	165	356	44	32	-	-	54	178	178	690	690		
LeTourneau WORKHORSE & 240-C	381	165	356	63	38	-	-	66	178	178	690	690		
LeTourneau Super Gorilla (Hull 219 - "Gorilla V")	584	191	559	108	38	-	-	121	279	279	690	690		
LeTourneau Super Gorilla & Super Gorilla XL	584	191	559	115	38	-	-	92	279	279	690	690		
MSC CJ70-150-MC (lower part)	650	210	600	95	70	-	-	100	285	285	690	690	7587	
MSC CJ70-150-MC (top part inner chord)	650	210	600	82,5	60	-	-	90	285	285	690	690	7587	
MSC CJ70-X150-A (lower part)	640	210	600	95	70	-	-	100	285	285	690	690	7587	
MSC CJ70-X150-A (top part inner chord)	640	210	600	82,5	60	-	-	90	285	285	690	690	7587	
MSC CJ50 (W-Larissa)	610	150	520	24	24	-	-	0	257	257	690	690	5969	
MSC CJ50 (Tam Dao)	640	150	520	18	18	-	-	0	257	257	690	690	5969	
MSC CJ50-X100-MC (lower part)	550	160	520	55	40	-	-	60	235	235	690	690	4618	
MSC CJ50-X100-MC (top part inner chord)	550	160	520	55	40	-	-	60	235	235	690	690	5938	
MSC CJ50-X80SJ (lower part)	550	152,4	520	55	40	-	-	60	238,8	238,8	690	690	4948	
MSC CJ50-X80SJ (top part inner chord)	550	152,4	520	55	40	-	-	60	238,8	238,8	690	690	5938	
MSC CJ46-X100-D (lower part)	540	150	520	28	28	-	-	0	193	193	690	690	4084	
MSC CJ46-X100-D (top part inner chord)	540	150	520	28	28	-	-	0	193	193	690	690	5027	
MSC CJ46-X100-D COSL (lower part)	540	150	520	32	32	-	-	0	193	193	690	690	4084	
MSC CJ46-X100-D COSL (top part inner chord)	540	150	520	28	28	-	-	0	193	193	690	690	5027	
MSC NG-2500X (lower part)	400	120	356	30	25	-	-	30	143	143	690	690	2827	
MSC NG-2500X (top part inner chord)	400	120	356	35	25	-	-	35	143	143	690	690	3142	
MSC NG-1700 (Bima)	391,5	95	355,6	18,5	18,5	-	-	0	152,5	152,5	690	500	5670	



Strength interaction equations (for mapping of the variables; see A.12.6.3.3) as given by Equation (F.3-4):

$$\left[\left(\frac{M_y}{M'_{py}} \right)^\xi + \left(\frac{M_z}{M'_{pz}} \right)^\xi \right]^{1/\xi} \leq 1,0 \tag{F.3-4}$$

where, for $(P/P_y) < 1,0$:

$$M'_{py} = M_{py} \left[1 - \left(\frac{P}{P_y} \right)^{1,45} \right]$$

when $M_z \geq 0,0$: $\xi = 1,8 + 2,7 \left(\frac{P}{P_y} \right) + 2,8 \left(\frac{P}{P_y} \right)^2 - 5,6 \left(\frac{P}{P_y} \right)^3$ and

$$M'_{pz} = M_{pz} \left[1 - \left(\frac{P}{P_y} \right)^{1,12} \right]^{1/1,12}$$

when $M_z < 0,0$: $\xi = 1,8$ and either

for $1,0 > (P/P_y) > 0,25$: $M'_{pz} = -M_{pz} \left[1 - \left(\frac{P}{0,75P_y} - \frac{1}{3} \right)^{1,45} \right]$

for $0,25 \geq (P/P_y)$: $M'_{pz} = -M_{pz}$

When $(P/P_y) \geq 1,0$ the member has failed.

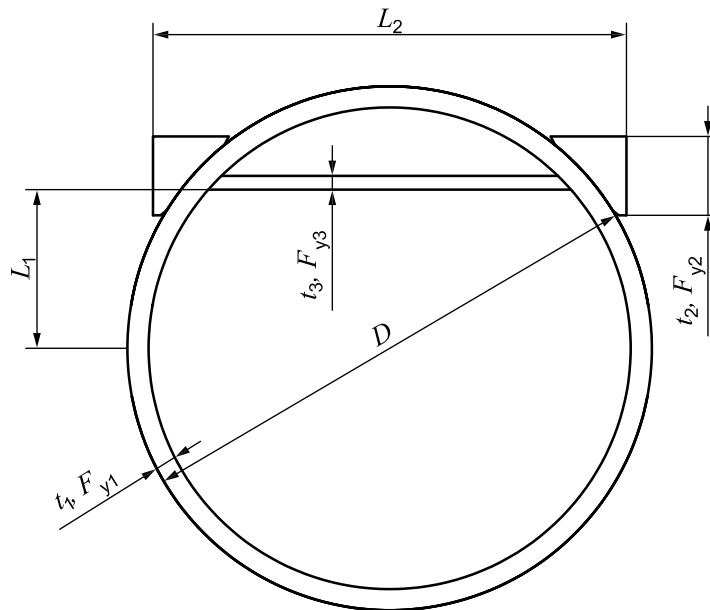
Key

- P chord member axial force
- P_y chord member axial strength
- M_y and M_z local y- and z-axis bending moments
- M_{py} and M_{pz} local y- and z-axis bending strengths
- M'_{py} and M'_{pz} adjusted local y- and z-axis bending strengths used in simplified interaction equations
- ξ interaction equation exponent for biaxial bending used in simplified interaction equations

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

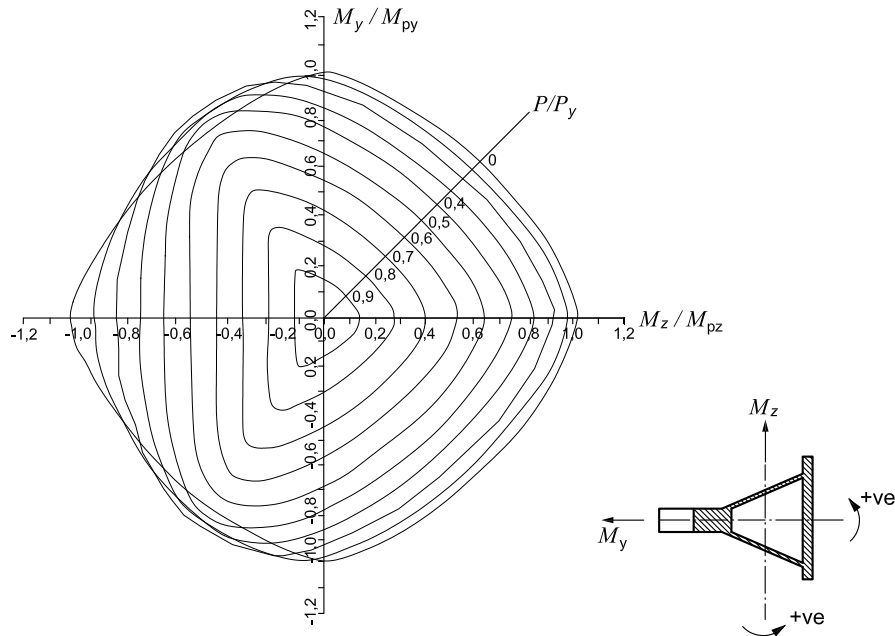
Legend: Chord dimensions



All dimensions are in millimetres; Yield stresses are in MPa

Design	D	t ₁	L ₁	L ₂	t ₂	t ₃	Stress			Bay Ht	Y _{ena}	Y _{cos}	E		
							F _{y1}	F _{y2}	F _{y3}						
Levingston 011-C	914	29	305	906	127	0	483	621	0	4826	84	100	16		
		33									75	90	15		
Levingston 111	1016	32	305	1047	127	0	690	690	0	4877	73	73	0		
		35									68	68	0		
Mitsui JC-300 (Key Hawaii)	1016	32	305	1046	127	0	690	690	0	5650	78	78	0		
		34											0		
		35									66	66	0		
Mitsui 1-off (Key Bermuda)	1016	29	305	1046	127	0	690	690	0	4672			0	Most of leg	
		29	305	1046	127	0	690	690	0	5050			0	Towage Section	
		32									5050	73	73	0	"
		36									5050	66	66	0	"
Hitachi Drill-Hope	762	30	190	882	127	0	690	690	0	5500	57	57	0		
		32									55	55	0		
Hitachi C-150 (Ile Du Levant)	762	30	190	890	130	0	690	690	0	5500	60	60	0		
Hitachi K1040/44/45	900	30	300	882	127	0	690	690	0	4800	77	77	0	Btm 2 bays	
		30									5090	77	77	0	Rest of leg
		35									5090				
		42									5090	60	60	0	
Hitachi K1060 (Sagar Lakshmi)	900	30	300	854	127	13	690	690	690	5260	84	84	0		
		31											0		
		32											0		
		34									77	77	0		
Robco 350-C	876	29	292	881	127	0	690	690	0	5461	83	83	0	Btm 3 bays	
		38									68	68	0	"	
		29									89	89	0	Rest of Leg	
		32									82	82	0	"	

where Y_{ena} is the distance from the origin to the elastic neutral axis;
 Y_{cos} is the distance from the origin to the centre of squash (plastic neutral axis);
 E is the distance between the elastic and plastic neutral axes.



Strength interaction equations (for mapping of the variables; see A.12.6.3.3) as given by Equation (F.3-5):

$$\left\{ \left(\frac{M_y}{M'_{py}} \right)^\xi + \left(\frac{M_z / M_{pz} - K}{M'_{pz} / M_{pz} - K} \right)^\xi \right\}^{1/\xi} \leq 1,0 \tag{F.3-5}$$

where, for $(P/P_y) < 1,0$:

$$K = -0,8 \left(\frac{P}{P_y} \right) + 0,4 \left(\frac{P}{P_y} \right)^2 + 0,4 \left(\frac{P}{P_y} \right)^3 \text{ and}$$

$$M'_{py} = M_{py} \left\{ 1 - \left(\frac{P}{P_y} \right)^{2,1} \right\}$$

When $(M_z/M_{pz}) \geq K$: $M'_{pz} = M_{pz} \left\{ 1 - \left(\frac{P}{P_y} \right)^{1,45} \right\}$ and
 $\xi = 1,45$

When $(M_z/M_{pz}) < K$: $M'_{pz} = -M_{pz} \left\{ 1 - \left(\frac{P}{P_y} \right)^{1,04} \right\}^{1/1,04}$ and
 $\xi = 1,45 + 2,35 \left(\frac{P}{P_y} \right) + 4,7 \left(\frac{P}{P_y} \right)^2$

When $(P/P_y) \geq 1,0$, the member has failed.

- Key**
- P chord member axial force
 - P_y chord member axial strength
 - M_y and M_z local y- and z-axis bending moments
 - M_{py} and M_{pz} local y- and z-axis bending strengths
 - M'_{py} and M'_{pz} adjusted local y- and z-axis bending strengths used in simplified interaction equations
 - ξ interaction equation exponent for biaxial bending used in simplified interaction equations
 - K major axis moment limiting value used to define simplified interaction equation exponent; it is the value of M_z/M_{pz} at which M_y/M_{py} is a maximum.

Figure F.3-4 — Interaction equations/curves for triangular chords with single racks

Table F.3-4 — Data for triangular chords with single rack

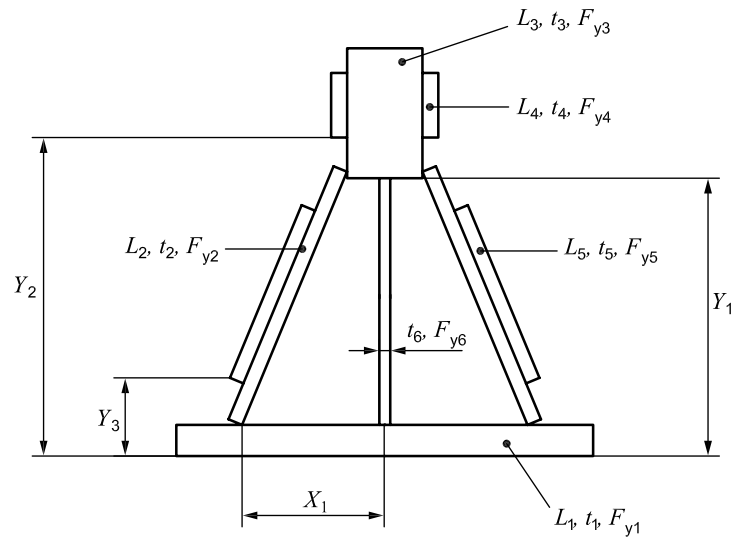


Table F.3-4 (continued)

All dimensions are in millimeters, Yield Stresses are in MPa

Design	Yield Stress																			
	L_1	L_2	L_3	L_4	L_5	L_6	X_1	Y_1	Y_2	Y_3	F_{V1}	F_{V2}	F_{V3}	F_{V4}	F_{V5}	Bay Ht	Y_{ena}	Y_{cos}	E	
MarLet Standard (3/4" side plates)	711	51	466	19	213	127	0	0	0	0	0	483	483	587	0	0	3408	259	279	20
MarLet Standard (7/8" side plates)	711	51	466	22	213	127	0	0	0	0	0	483	483	587	0	0	3408	259	279	20
MarLet Standard (1" side plates)	711	51	466	25	213	127	0	0	0	0	0	483	483	587	0	0	3408	260	279	19
MarLet Standard (1.5" side plates)	711	51	466	38	213	127	0	0	0	0	0	483	483	587	0	0	3408	262	279	17
MarLet Standard + side stiffeners	711	51	466	19	213	127	0	0	127	25	0	211	483	483	587	0	483	260	279	19
MarLet Std 116 (1"x4" rack stiffeners)	711	51	466	19	213	127	102	25	0	0	0	236	457	524	483	0	3408	278	296	18
MarLet 116 North Sea (1"x4"+1"x12" stiffns)	711	51	466	19	213	127	102	25	305	25	0	236	457	524	483	0	3408	276	291	15
MarLet 116 (1"x4"+1.5"x12" stiffns)	711	51	466	19	213	127	102	25	305	38	0	236	457	524	483	0	3408	277	291	14
MarLet 116 Juneau (2"x4"+1"x12" stiffns)	711	51	466	19	213	127	102	51	305	25	0	248	457	524	483	0	3408	290	304	14
MarLet Gorilla (150-88)	813	76	573	57	222	140	0	0	0	0	0	483	483	620	0	0	5113	302	323	21
MarLet Super 300	813	76	607	38	222	140	0	0	0	0	0	483	483	620	0	0	5113	298	323	25
MarLet 300 Slant	813	76	607	38	222	140	0	0	305	51	0	296	483	483	620	0	483	327	346	19
LeTourneau 150 (3/4" side pl)	711	64	441	38	213	127	0	0	0	0	0	414	414	414	0	0	2556	245	245	0
LeTourneau 150 (1.125" side pl)	711	51	466	19	213	127	0	0	0	0	0	414	414	620	0	0	2556	259	302	43
LeTourneau 150 (1.5" side pl)	711	51	466	29	213	127	0	0	0	0	0	414	414	620	0	0	2556	259	303	44
LeTourneau 46,47	711	51	466	38	213	127	0	0	0	0	0	414	414	620	0	0	2556	262	298	26
LeTourneau 4,9	559	44	432	13	178	89	0	0	0	0	0	166	432	0	0	0	3408	224	224	0
Mitsubishi MD-T76J	750	50	574	25	225	125	0	0	0	0	0	687	687	687	0	0	3430	286	286	0
Gusto 1-off: (Maersk Endeavour)	800	60	592	30	283	127	0	0	0	0	0	620	620	620	0	0	3456	315	315	0
	800	90	534	40	283	127	0	0	0	0	0	620	620	620	0	0	4800	284	284	0
	800	110	488	50	283	127	0	0	0	0	0	620	620	620	0	0	4800	255	255	0
	800	76	535	38	279	127	0	0	0	0	0	690	690	690	0	0	4800	244	244	0
	800	64	549	38	279	127	0	0	0	0	0	690	690	690	0	0	4539	268	268	0
	800	51	562	32	279	127	0	0	0	0	0	690	690	690	0	0	4539	282	282	0
	800	51	562	29	279	127	0	0	0	0	0	690	690	690	0	0	4539	297	297	0
BMC 1-off design (Trident 7)	711	38	356	19	279	127	0	0	0	0	0	690	690	690	0	0	4539	297	297	0

ADDITIONAL DATA - THESE SECTIONS WERE NOT INCLUDED IN THE ANALYSES USED TO DERIVE Figure F.3-4 AND THEREFORE Figure F.3-0034 SHOULD NOT BE USED FOR THEIR EVALUATION

LeTourneau Super 116	711	51	460	32	222	127	-	-	-	-	-	-	-	-	-	-	587	587	690	-	587	
No Strapping - Hull 222 & Up	711	51	460	32	222	127	-	-	-	254	32	-	-	-	-	-	180	587	587	690	-	587
With Strapping																						
leTourneau Super 116E	711	51	460	32	222	127	-	-	-	-	-	-	-	-	-	-	587	587	690	-	-	
127 mm Rack Plate	711	51	392	38	286	165	-	-	-	-	-	-	-	-	-	-	690	690	690	-	-	
165 mm Rack Plate																						

Where Y_{ena} is the distance from the origin to the elastic neutral axis; Y_{cos} is the distance from the origin to the centre of squash (plastic neutral axis); E is the distance between the elastic and plastic neutral axes.

Annex G (informative)

Contents list for typical site-specific assessment report

This annex provides an outline for the contents of a site-specific assessment report.

Table G.1 — Jack-up and site

Element	
Jack-up name	
Jack-up type	
Operator	
Site name	
Latitude	
Longitude	

Table G.2 — Data check

Data		Does it exist?
Jack-up data	Is jack-up in class?	
	Soils data	
	Prior experience	
	Adjacent infrastructure	
Metocean data		
Arrangement at site	Jack-up heading (required if using directional data)	
	Water depth	
	Hull clearance above LAT	
	Conductor top tension/support mechanism	
	Conductor diameter and number	
Earthquake data		
Accidental situations		
Operator requirements (e.g. required airgap)		
Exposure level		
Agreed consequence class		
Life safety category		

Table G.3 — Site-specific assessment results summary

Site-specific assessment results summary	Is it acceptable?
Is jack-up suitable for the specified operation at the site and time of year?	
Are there specific seasonal and/or operational restrictions or limitations	
Minimum leg reserve above upper guide	
Foundation fixity used?	
Preload OK	
Foundation OK	
Member strength OK	
Overturning OK	
Possible infrastructure interaction	
Hull displacement	
Earthquake	
Accidental	

Table G.4 — Jack-up data

Jack-up data		Value
Length		
Breadth		
Depth		
Standard leg length		
No of legs		
No of chords/leg (1 to 4)		
Longitudinal leg spacing		
Transverse leg spacing		
Chord spacing	Chord spacing	
	Reference point for chord spacing, e.g. pitch points	
Weight of one leg including spudcan, including permanent ballast, but excluding water ballast and buoyancy		
Weight of one spudcan, including permanent ballast, but excluding water ballast and buoyancy		
Are legs (not spudcans) free-flooding?		
Type of holding system (jacks or fixation system)	Number of jacks per leg	
	Jack holding strength (jacking)	
	Jack holding strength (design maximum holding)	
	Jack holding strength (preload holding)	
	Jack holding strength (ultimate)	
Light ship		
Movable fixed load		
Variable load		
Total maximum hull weight		
Total minimum hull weight		
Overall hull centre of gravity (and tolerance where applicable)		
Total available preload		
Type of preload procedure (e.g. one leg at a time)		
Maximum preload spudcan reactions at the seabed using chosen preload method (including leg/spudcan weight and buoyancy)	Bow leg	
	Port leg	
	Starboard leg	
	Other legs	
Spudcan diameter		
Spudcan height		
Spudcan volume		
Maximum bearing area of spudcan		
Distance from spudcan maximum bearing area to tip		
Advertised operating water depth		
Designer		
Class/type		
Classification society		

Table G.5 — Arrangements at site

Arrangements at site	Value
Installed leg length	
Distance from keel to top of upper guide	
Hull clearance above LAT	
Water depth	
Expected penetration with full preload	
Reserve of leg	

Table G.6 — Metocean conditions

Metocean conditions ^a		Value/answer
50 year independent extremes or 100 year joint probability?		
Partial action factor		
Has directional metocean data been used?		
Has seasonal metocean data been used?		
Water depth		
Wave details	Maximum wave height	
	Associated wave period	
	Type of associated wave period supplied (intrinsic or apparent)	
	Significant wave height	
	Peak period	
	Type of peak wave period supplied (intrinsic or apparent)	
	Wave crest height	
Wind speed (at 10 m above water level, collinear with wave)	1 hour wind speed	
	1 minute wind speed (required)	
	3 second gust	
Surge		
Tide		
Reserve on hull clearance		
Hull elevation above LAT		
Expected storm settlement		
Other allowances, e.g. reservoir settlement		
Current	Surface current (collinear with wind and wave)	
	Bottom current (collinear with wind and wave)	
	Current profile details	
Marine growth	Profile	
	Predeployment marine growth profile	
Are there operational restrictions (e.g. variable load limits, heading, air gap, leg/guide location)?		
Are there specific operator requirements that may affect the suitability for the site?		
^a The contents of this table should be expanded as necessary to account for directional and seasonal data.		

Table G.7 — Site investigation

Site investigation	Does it exist?	Year
Bathymetry survey		
Shallow seismic survey		
Intrusive site investigation (soils boring/CPT)		
Magnetometer survey		

Table G.8 — Site hazards

Site hazards	Description
Pipeline hazards	
Adjacent structures	
Site move-on hazards, e.g. mudslides, sand waves, footprints, seabed slope	

Table G.9 — Soils

Soils	Value/description	
Distance from location of survey		
General soils description		
Seabed slope/features		
Profile details		
Variability over area		
Confidence in data		
Previous experience in the field		
Load penetration curve		
Range of predicted penetrations after preloading		
Is there a punch-through potential during installation?	Yes/No	
	Method for mitigating hazards	
Is there a risk of punch-through or significant settlement if the foundation reactions exceed capacity developed by preloading?	Yes/No	
	Method for mitigating hazards	
Does the predicted penetration curve show potential for punch-through or significant settlement (precipitous settlement) if the foundation reactions exceed those assessed?		
NOTE For information only: there is no acceptance criterion.		
Previous spudcan holes?	Yes/No	
	Method for mitigating hazards	
Other geotechnical hazards?	Yes/No	
	Method for mitigating hazards	
Is there scour potential?	Yes/No	
	Method for mitigating hazards	

Table G.10 — Analysis path/route/assumptions

Analysis path/route/assumptions

Table G.11 — Spudcan fixity used in analysis

Spudcan fixity used in analysis	Value
Initial stiffness (for each spudcan, if soils differ)	Rotational
	Lateral
	Vertical
Ultimate capacity	Rotational
	Lateral
	Vertical

Table G.12 — Earthquake analysis

Earthquake analysis	Description
Does site fall below the ISO 19905-1 cut-off (ISO 19901-2 seismicity level 2, or level 1 under the conditions specified in 10.7)?	
Was an earthquake analysis performed?	
Source of the earthquake data	
What vertical ground motions were used (in many cases the critical condition)?	
Was vertical spectrum some ratio of lateral ground motion spectrum, and if so, how was it derived?	
Is spudcan fixity different from metocean analysis, and if so what value was used?	
Was linear analysis sufficient to prove acceptability of site?	
Describe non-linear analysis, if used	
What was limit on vertical settlement, and differential settlement?	
Were effects on platform that jack-up was working over considered, if applicable (e.g. effects of interaction due to lateral motions or vertical settlement)?	

Table G.13 — Accidental situations

Accidental situations	Description
Were any accidental situations assessed (e.g. collision)?	
What were the results of the analyses?	

Table G.14 — Intermediate results

Intermediate results		Value
Natural period (with fixity and P- Δ effects)		
DAF	If SDOF, give DAF on BS, $K_{DAF,SDOF}$	
	Stability of DAF with simulation duration (2-stage)	
	Stability of static and dynamic MPME (1-stage)	
	If random, give DAFs on BS and OTM, $K_{DAF,RANDOM}$	
Factored wind, wave/current, inertial BSs and OTMs		
<p>The DAF calculated in the SDOF analogy ($K_{DAF,SDOF}$) should not be directly compared to the DAF determined with a stochastic wave assessment ($K_{DAF,RANDOM}$). Because the method of determining the relevant inertial loadset is different, the same value of $K_{DAF,SDOF}$ and $K_{DAF,RANDOM}$ produces different total global responses; see Figure A.10.5-1.</p>		

Table G.15 — Analysis results (utilization checks)

Analysis results (utilization checks)		Value
Preload	Potential for punch-through when preloading?	
	Mitigations proposed	
Foundation	Bearing check	
	Sliding check	
	Magnitude of additional penetration under storm loads	
	Potential for punch-through when elevated?	
	Mitigations proposed	
Overturning		
Chord member strength		
Horizontal member strength		
Diagonal member strength		
Jacks/fixation system		
Results of earthquake analysis (if applicable)		

Annex H (informative)

Regional information

H.1 General

This annex contains provisions for a limited number of regions; the content has been developed by ISO/TC 67 experts from the region or country concerned to supplement the provisions of this part of this International Standard. Each provision can be considered to constitute the additional information required for regional implementation for the particular region or country defined. The regional information may provide regional and national data that can include regional environmental conditions and local assessment and operating practices. The regulatory framework may be explained but neither regulatory requirements nor reference to specific legislation is included in this International Standard.

H.2 Norway

H.2.1 Description of region

The provisions in H.2 applies to areas under Norwegian jurisdiction.

H.2.2 Regulatory framework

The content of H.2 is laid down pursuant to the Norwegian Act 29, November 1996, No. 72, relating to petroleum activities.

H.2.3 Technical requirements

H.2 contains additional requirements for site-specific assessment of jack-ups in Norwegian waters. The following provisions are in addition to, or an alternative to, those specified in the appropriate referenced subclauses.

- 5.3 d) The site-specific risks, such as collision risk and geohazard, shall be evaluated. ALS actions shall be defined based on a site-specific risk analysis.
- 5.5.4 and 8.8 An action factor of 1,25 should be used for jack-ups in L1, in combination with environmental conditions with an annual probability of exceedance 10^{-2} . An action factor of 1,25 should be applied for jack-ups in L2 in manned conditions. An action factor of 1,15 can be used for evacuated jack-ups in L2, in combination with environmental conditions an annual probability of exceedance 10^{-2} .
- 6.4 100 year joint probability metocean data shall be used for extreme storm event assessments (ULS assessment) for jack-ups in Norwegian waters. If reliable 100 year site-specific joint probability data do not exist, a combination of 100 year waves, 100 year wind and 10 year current can be applied.
- 6.5 The relaxation “For sites where previous operations have been performed by jack-ups...” shall be applied only if the previous jack-up has been evaluated according to this part of ISO 19905 and has an equal or a more severe performance than the jack-up in question.
- 6.5 Soil investigations after installation are not considered good practice.
- 8.8.1 When checking the ULS, the SLS, the ALS and the FLS, the action factors shall be used according to Table H.2.3-1 for L1. For L2 the action factor 1,25 can be reduced to 1,15 for environmental actions.

Table H.2.3-1 — Partial action factor for the limit states controls for site-specific evaluations

Limit state	Load case	Action due to fixed load G_F	Action due to variable load G_V	Environmental action E^a
SLS	—	1,0	1,0	1,0
ULS	a ^b	1,2 ^c	1,3	0,7
ULS	b	1,0	1,0	1,25
ALS	Abnormal effects ^d	1,0	1,0	1,0
ALS	Damaged conditions ^e	1,0	1,0	1,0
FLS	—	1,0	1,0	1,0

a Earthquake shall be handled as environmental action within the limit state design for ULS and ALS.
b For fixed and/or variable loads, G_F and G_V , an action factor of 1,0 shall be used where this gives the most unfavourable action effect.
c If the actions do not have a well defined upper limit, e.g. uncertainty in the fixed loads, the coefficient 1,2 should be increased to 1,3.
d Actions with annual probability of exceedance of 10^{-4} .
e Environmental actions with annual probability of exceedance of 10^0 (one year return period).

The ULS load case “a” and the FLS are normally covered by the RCS class certificates. If the facility is used for more than five years on the same location, a site-specific fatigue analysis shall be performed.

- 10.5.2 The SDOF-method can be used only when it is conservative.
- 10.7 An annual probability of exceedance of 10^{-2} in ULS and 10^{-4} in ALS shall be used for earthquakes.
- 13.6 For L1 platforms, the hull elevation shall be sufficient to clear the crest elevation of waves with an annual probability of exceedance 10^{-4} . Alternatively, it may be documented that the platform has sufficient strength to withstand metocean actions (for the ALS) with an annual probability of exceedance 10^{-4} .
- 13.11 An annual probability of exceedance 10^{-2} shall be used for temperature.
- Annex A Statements related to return periods and action factors are covered by the additional requirements above.
- A.6.4.2.2 A factor of 1,9 can be used between the individual extreme wave height (H_{max}) and the significant 100 year wave.
- A.7.3.1.1 If site-specific joint probability data does not exist, a combination of 100 year waves, 100 year wind and 10 year current can be applied.
- A.7.3.2.5 The marine growth should be in accordance with ISO 19901-1:2005, Table C.2, if the jack-up is on one location for a long time. The default value given in A.7.3.2.5 shall only be used when an effective antifouling system is in place or systematic cleaning is to be performed.
- A.10.5.2.2.2 and Table A.10.4-1 The damping should be based on measured values from the actual jack-up, or from jack-ups with similar spudcans, foundation and leg to hull connection system. If measurements do not exist, a damping of 2 % to 4 % should be used for FLS analysis. Specific evaluations can be needed to justify the values adopted.
- A.12.2.2 The maximum value of the yield strength used in the analyses should not be greater than the ultimate tensile strength divided by 1,2.

- A.12.5 and A.12.6 The partial resistance factors should be increased by a factor of at least 1,05.
- Annex B Several of the partial factors are replaced by the factors given above.

H.2.4 Technical commentary

The national body responsible for preparing Offshore Norway's regional annex is the Petroleum Safety Authority Norway. This organization is the contact point for any questions arising from the contents of this annex.

H.2.5 Additional national requirements

The regulations relating to health, environment and safety in the petroleum activities (the framework regulation), laid down by Royal Decree 31 August 2001 stipulates in § 3 that mobile facilities registered in a national register of shipping, and that follow a maritime operational concept, relevant technical requirements contained in rules and regulations of the Norwegian Maritime Directorate in the form following the 2007 regulations and later amendments, together with supplementary classification regulations issued by Det Norske Veritas, or international flag state rules with supplementary classification rules achieving the same level of safety, may be used as an alternative to technical requirements laid down in the facility regulation or pursuant to the Petroleum Act.

Facilities not using the option in the framework regulation § 3, shall comply with the Petroleum Safety Authority Norway: Regulations relating to design and outfitting of facilities, etc., in the petroleum activities (the facility regulation).

Independent of flag state, the technical requirements in the Norwegian Maritime Directorate: Regulations of 4 September 1987, no. 856, concerning construction of mobile offshore units are valid for facilities using the option in the framework regulation § 3. Units used in Norway shall also comply with the technical requirements of Det Norske Veritas standard DNV-OS-C104^[A.11.3-1].

H.3 US Gulf of Mexico

H.3.1 Description of region

The geographical extent of the region are the waters of the Gulf of Mexico that fall within the United States exclusive economic zone (EEZ), which is generally the portion of the Gulf of Mexico north of 26° N, as shown on Figures H.3-1 and H.3-2, and which includes the shallow water lease blocks shown on Figures H.3-2.

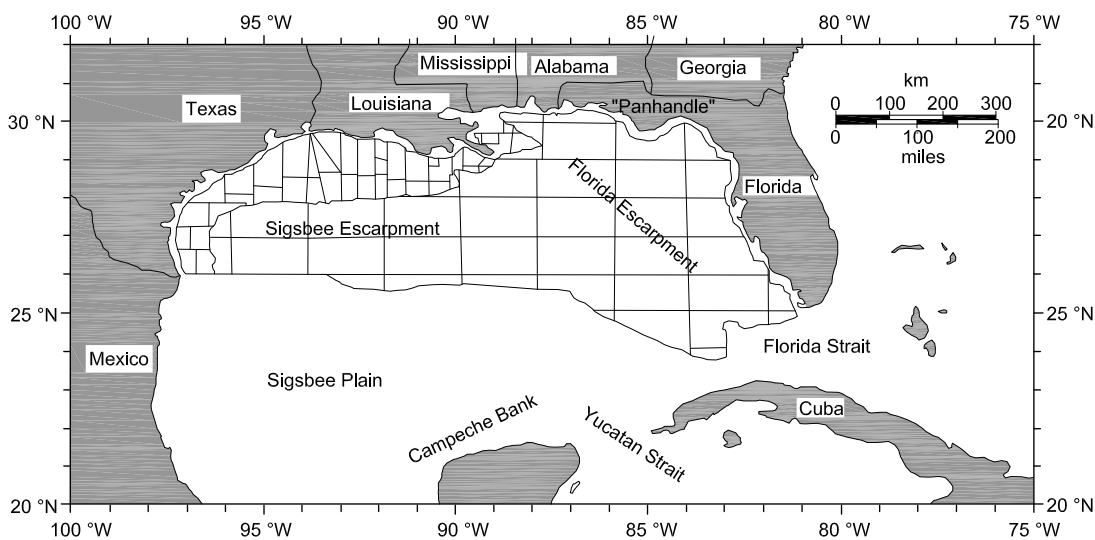


Figure H.3-1 — Northern Gulf of Mexico — Outer continental-shelf and deep water US lease areas^[H.3-1]

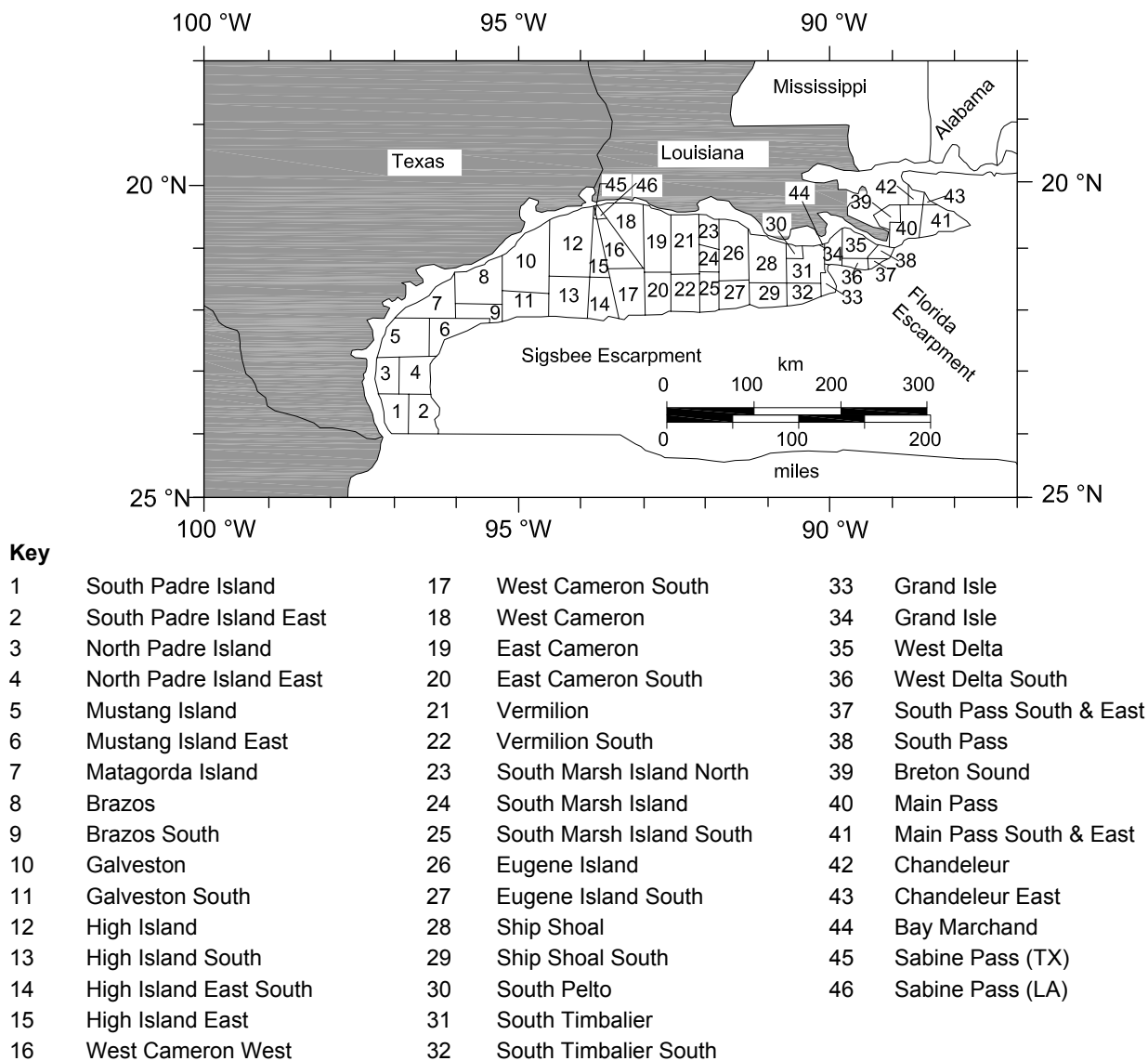


Figure H.3-2 — Northern Gulf of Mexico: Inner continental-shelf US lease areas^[H.3-1]

H.3.2 Regulatory framework

The U.S. Bureau of Ocean Energy Management, Regulation, and Enforcement (BOEMRE), formerly the U.S. Mineral Management Service, has jurisdiction over the operations of MOUs on the U.S. Outer Continental Shelf. Supplementing its regulations, BOEMRE has issued requirements for the operation of jack-ups, including site assessments, in its “Notice to Leaseholders” (NTLs). The current BOEMRE regulations and NTLs for site assessment should be consulted when applying this part of ISO 19905, including this clause.

H.3.3 Metocean conditions

H.3.3.1 General

As described in ISO 19901-1, the climate in the northern Gulf of Mexico ranges from tropical to temperate. Summer wind and wave conditions are generally benign, with warm temperatures and high relative humidity. There are occasional light squalls and thunderstorms. The extreme wind and wave climate in the Gulf is dominated by hurricanes in the summer season and the passage of non-tropical frontal systems in the winter season. Swell is not a major factor except when associated with a hurricane. Waves tend to be correlated with winds (either hurricane or winter storm) and temporarily strong currents can be associated with storm systems, although there are also prevailing circulation currents even in the shallow Gulf.

The hurricane season officially runs from the beginning of June to the end of November, and on average three tropical storms can be expected to form in or enter the region each year. These storms can originate in the Gulf of Mexico, the Caribbean Sea or in the North Atlantic Ocean.

The National Hurricane Center monitors the gestation and development of all tropical disturbances in the Atlantic region. After tropical storm development they fly hurricane hunter aircraft into the storms to gather data. Information is released to the public on the formation, development, the expected track and speed and wind speeds. Other public and private organizations develop forecast metocean conditions from these data. Government bodies, operators and jack-up owners can use these data to plan for the evacuation of offshore personnel. This also allows the operators and owners time to prepare jack-ups for the storm event prior to evacuation. The standard practice is to evacuate all personnel prior to the arrival of severe weather.

The exception to this situation is when a sudden TRS forms within the Gulf (sudden hurricane). In this case, it might not be possible to evacuate personnel; however, historical metocean data indicates that tropical storms or hurricanes that develop within the Gulf are much less severe than the major hurricanes that develop in the Atlantic basin and migrate into the Gulf. Even in the case of a sudden hurricane, there is sufficient time to make safe the well and prepare the jack-up for severe weather.

H.3.3.2 Metocean conditions and their assessment

H.3.3.2.1 General

An L1 jack-up shall be assessed to metocean data applicable to the season of operation using either the 50 year extreme with a partial action factor of 1,15 or the 100 year joint probability metocean data with a partial action factor of 1,25.

An L2 jack-up shall be assessed for the situation that can be reached prior to evacuation being effected. This assessment shall be to L1 criteria using the 50 year independent extremes or 100 year joint probability data for hurricanes that can reach the site prior to evacuation being effected. This annex provides 50 year 48 hour notice sudden hurricane data for the northern Gulf of Mexico. Relevant data and criteria are given in H.3.3.2.2 and H.3.3.2.3. This annex also includes additional requirements for L2 jack-ups; see H.3.3.2.4. OTC 17879^[H.3-2] gives the background and derivation of these data.

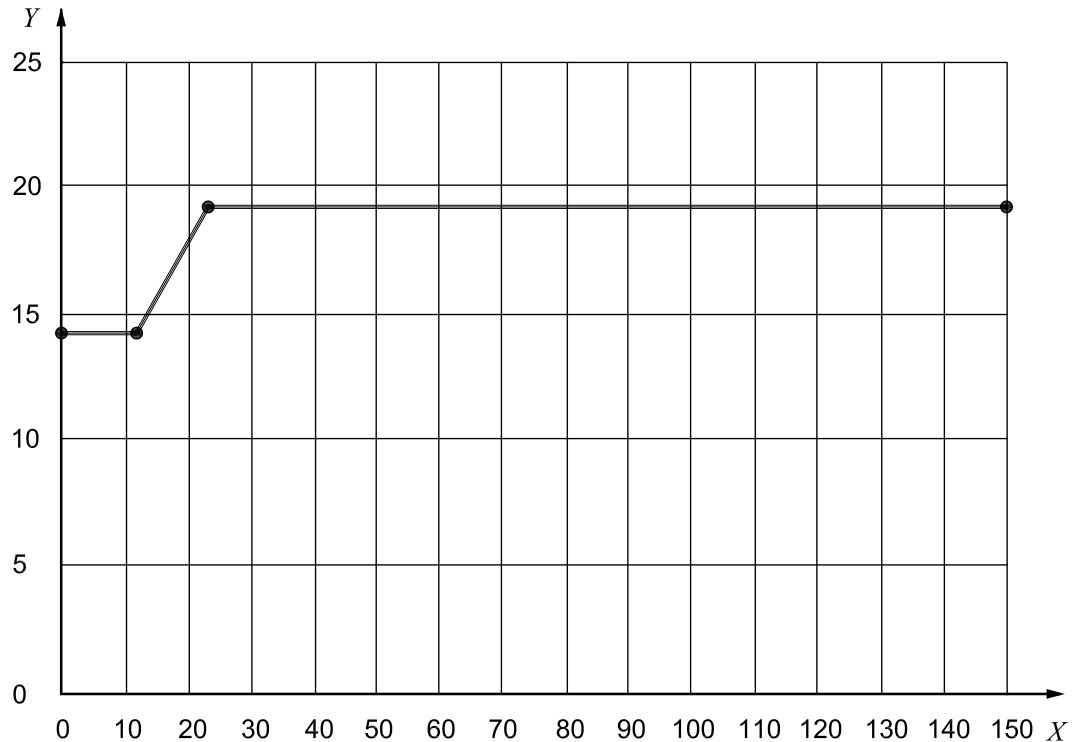
An L2 jack-up shall also be assessed as an unmanned unit for the post evacuation case (see H.3.3.2.5).

Other requirements for hurricane season are given in H.3.3.3.

An L3 jack-up shall be assessed to criteria agreed between the jack-up owner and the operator.

H.3.3.2.2 Hull elevation during hurricane season

With reference to 5.4.5 and 13.6, the assessor shall consider the possibility of wave impingement on the hull. The hull elevation considered in the site-assessment shall be appropriate for the water depth, spring tide, and the expected maximum wave crest height and storm surge due to the 100 year return period hurricane. The hull elevation shall also include an allowance for any settlement predicted by the post evacuation assessment. In the absence of a site-specific hull elevation assessment, the curve given in Figure H.3-3 may be used.

**Key**

X water depth, expressed in metres

Y minimum recommended hull elevation above LAT, expressed in metres

Figure H.3-3 — Recommended hull elevation above LAT for hurricane season

H.3.3.2.3 Assessment case

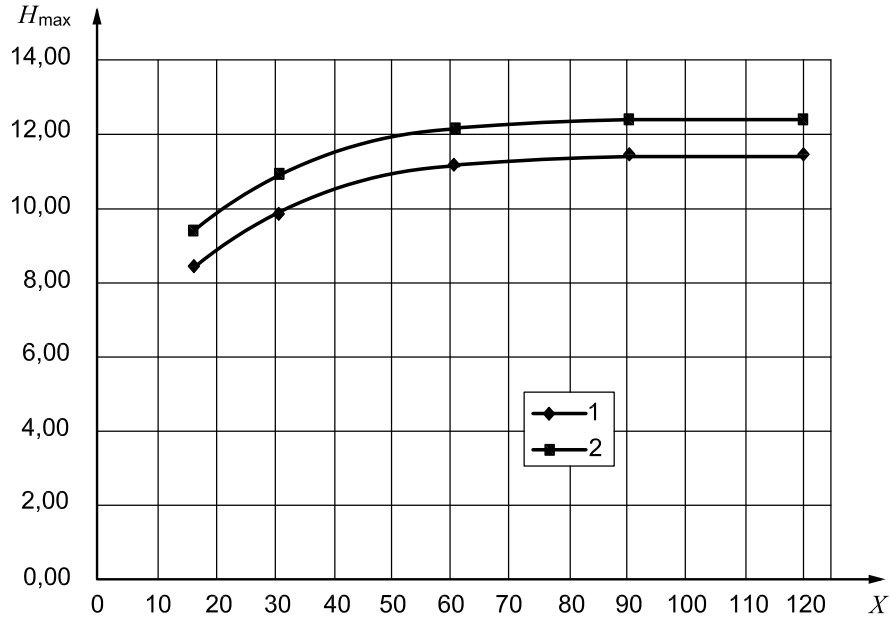
When it can be demonstrated that the jack-up can be placed in storm survival mode and evacuation effected within 48 hours (see 5.5.1, 5.5.2, 5.5.4, A.6.4.1), the jack-up shall be assessed to the 50 year return period 48 hour sudden hurricane conditions using the action and resistance factors given in this part of ISO 19905, as summarized in Annex B. When an effective evacuation cannot be accomplished within 48 h, site-specific metocean data shall be used for the necessary longer evacuation time. Similarly, when it can be demonstrated that a lesser evacuation time can be assured, a reduced time may be used to determine the revised metocean data.

48 hour sudden hurricane metocean data for the assessment case are given in Figures H.3-4 and H.3-5 and Tables H.3-1 and H.3-3. The significant wave height should be taken as $H_{\max}/1,75$. The wave periods given in the Table H.3-3 should be considered with a $\pm 0,5$ sec range unless a more detailed study indicates otherwise.

H.3.3.2.4 Contingency case

Additionally, the jack-up shall be assessed for a contingency case, using sudden hurricane conditions for a period 24 hours longer than that used for the assessment case, a metocean action factor $\gamma_{1,E}$ of 1,0 and the resistance factors given in this part of ISO 19905.

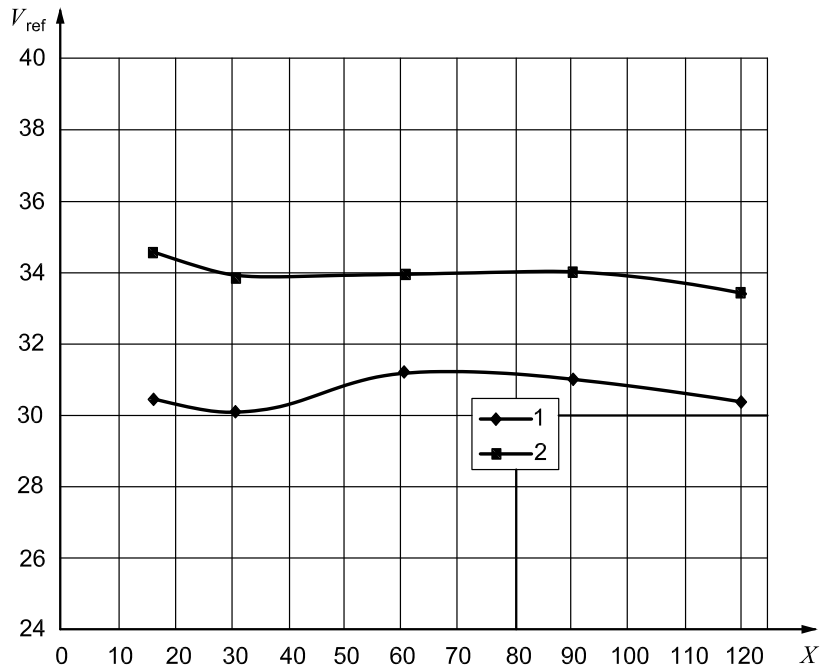
72 hour sudden hurricane metocean data for the contingency case are given in Figures H.3-4 and H.3-5 and Tables H.3-2 and H.3-4. The significant wave height should be taken as $H_{\max}/1,75$. The wave periods given in the Table H.3-4 should be considered with a $\pm 0,5$ second range unless a more detailed study indicates otherwise.



Key

- X water depth (including surge), expressed in metres
- H_{max} maximum wave height, expressed in metres
- 1 assessment case
- 2 contingency case

Figure H.3-4 — Maximum wave height for US Gulf of Mexico site assessments



Key

- X water depth (including surge), expressed in metres
- V_{ref} 1 min sustained wind speed, expressed in metres per second
- 1 assessment case
- 2 contingency case

Figure H.3-5 — Maximum wind speed for US Gulf of Mexico site assessments

Table H.3-1 — Assessment case current profiles

Water depth	Current at surface	Current at mid-depth ^a	Current at bottom of profile	Elevation of bottom of profile above mudline
m	m/s	m/s	m/s	m
15	1,22	1,13	1,04	3
30	1,03	0,93	0,86	4
60	0,78	0,73	0,68	5
90	0,72	0,67	0,63	25
120	0,64	0,60	0,56	55

^a Mid-point between surface and bottom of profile.

Table H.3-2 — Contingency case current profiles

Water depth	Current at surface	Current at mid-depth ^a	Current at bottom of profile	Elevation of bottom of profile above mudline
m	m/s	m/s	m/s	m
15	1,43	1,32	1,22	3
30	1,23	1,10	0,98	4
60	0,86	0,80	0,75	5
90	0,77	0,72	0,68	25
120	0,68	0,64	0,60	55

^a Mid-point between surface and bottom of profile.

NOTE 1 The current profiles in the above tables are defined from the surface to the bottom of profile elevation above the mudline defined in the right-most column. The current profile then decays linearly to the mudline.

NOTE 2 For water depths not defined, interpolate between values given.

Table H.3-3 — Assessment case wave periods

Water depth	$T_{p,i}$	T_{ass}
m	s	s
15	10,1	9,4
30	10,3	9,6
60	10,7	9,9
90	10,8	10,1
120	10,9	10,1

Table H.3-4 — Contingency case wave periods

Water depth m	$T_{p,i}$ s	T_{ass} s
15	11,0	10,2
30	10,9	10,1
60	11,0	10,3
90	11,1	10,4
120	11,1	10,3

H.3.3.2.5 Unmanned post-evacuation case

In accordance with 5.5.4, the jack-up shall also be considered for the unmanned post-evacuation case using criteria agreed upon between the jack-up owner and the operator. A typical assessment can proceed using one of the following approaches.

- a) Elastic analysis: Perform an assessment to the ULS requirements of this part of ISO 19905, but with the action and resistance factors set to 1,0.
- b) Plastic collapse (pushover) analysis: Create load cases for the environmental conditions selected for the post evacuation assessment using the calculation methodology in this part of ISO 19905. The component strength checks are replaced by a system strength check based on plastic collapse techniques; see 10.9. The effect of additional settlement should be included to assess the potential for collapse.

For both types of post evacuation analyses described above, the added P-Δ effect due to leg settlement shall be considered and a Step 3 displacement check shall be performed for the foundations.

H.3.3.3 Other requirements

H.3.3.3.1 Preloading

The maximum feasible preload reaction should normally be applied. This can require individual leg preloading, which is, in general, recommended. The preload shall be applied and held for a reasonable period after penetration has ceased. Frequently the holding period is from one hour to two hours for a typical Gulf of Mexico location. This guidance should be tempered with knowledge of the soils at the location. For instance, where punch-through potential exists, holding times should be increased.

H.3.3.3.2 Storm preparation

Sufficient time shall be allocated within the evacuation plan to place the jack-up in survival mode prior to evacuation, as described in the Marine Operations Manual (MOM).

Where possible, the lower-guide should be located at an optimal position.

Where required by the marine operations manual, the drill package shall be skidded to a storm position.

Consideration should be given to increasing the hull elevation to avoid wave impingement on the hull, or to reducing the hull elevation to lower the dynamic effect, or changing the hull elevation to reduce the potential for impingement on adjacent structures (see 5.4.7 and 13.10).

The conductor support requirements should not normally impede the placing of the jack-up into survival mode when this is prescribed by the MOM or other site-specific requirements. However, if the operator requires that the conductor is to remain supported during a storm, the resulting loads shall be considered in the assessment (see 8.8.7).

Bibliography

- [1] ISO 19901-3, *Petroleum and natural gas industries — Specific requirements for offshore structures — Part 3: Topsides structure*
- [2] ISO 19901-4:2003, *Petroleum and natural gas industries — Specific requirements for offshore structures — Part 4: Geotechnical and foundation design considerations*
- [3] ISO 19901-6, *Petroleum and natural gas industries — Specific requirements for offshore structures — Part 6: Marine operations*
- [4] ISO 19901-7:2005, *Petroleum and natural gas industries — Specific requirements for offshore structures — Part 7: Stationkeeping systems for floating offshore structures and mobile offshore units*
- [5] ISO/TR 19905-2:2012, *Petroleum and natural gas industries — Site-specific assessment of mobile offshore units — Part 2: Jack-ups commentary and detailed sample calculation*
- [6] ISO 19906, *Petroleum and natural gas industries — Arctic offshore structures*
- [7] Society of Naval Architects and Marine Engineers (SNAME), *Recommended practice for site specific assessment of mobile jack-up units*, Technical and Research Bulletin 5-5A, Rev 2, New Jersey, 2002
- [7.3-1] Det Norske Veritas, *Environmental Conditions and Environmental Loads*, DNV-RP-C205, Oslo, April 2010
- [7.3-2] GRUNDMEIER, B.L., CAMPBELL, R.B. and WESSELINK, B.D., *A Solution for Wind-Induced Vortex-Shedding Vibration of the Heritage and Harmony Platforms During Transpacific Tow*, Proc. 21st Offshore Technology Conf., OTC 6174, May 1989
- [7.3-3] BLEVINS, R.D. *Flow Induced Vibration*, Van Nostrand Reinhold International, 2nd Edition, 1990
- [12.1-1] American Petroleum Institute, *Stability Design of Cylindrical Shells*, API Bulletin 2U, June 2004
- [12.1-2] American Petroleum Institute, *Design of Flat Plate Structures*, API Bulletin 2V, 2004
- [12.1-3] Det Norske Veritas, *Buckling Strength of Plated Structures*, DNV RP-C201, Oslo, October 2004
- [12.1-4] Det Norske Veritas, *Buckling Strength of Shells*, DNV RP-C202, Oslo, October 2002
- [13.6-1] FORRISTALL, G.Z., *Wave Crest Heights and Deck Damage in Hurricanes Ivan, Katrina, and Rita*, Proc. 39th Offshore Technology Conf., OTC 18620, Houston, May 2007
- [A.1.1-1] The Institute for Petroleum, *Guidelines for the Analysis of Jack-up and fixed Platform Well Conductors*, July 2001
- [A.6.4-1] HOYLE, M.J.R., SMITH, S.F., AHILAN, R.V., HUNT, R.J. and MARCOM, M.R., *Further Work on the Effects of Nonlinear Wave Spreading and its Impact on Current Assessment Practice*, Proc. 41st Offshore Technology Conf., OTC 20297, May 2009
- [A.6.4-2] LEGGETT, J.M., BELLAMY, N.F., FOX, J.F. and SHEIKH, R., *A Recommended Approach for Deriving ISO-Compliant 10,000 Year Extreme Water Levels In the North Sea*, Proc. of the 26th Int. Conf. on Offshore Mechanics and Arctic Engineering (OMAE), OMAE-29559, San Diego, 2007
- [A.6.4-3] Det Norske Veritas, *Environmental Conditions and Environmental Loads*, DNV-RP-C205, Oslo, April 2010

- [A.6.4-4] CARTER, D.J.T., *Estimation of Wave Spectra from Wave Height and Period*, I.O.S. Report No. 135, 1982
- [A.6.5-1] RPS Energy, *Improved Guidelines for the Prediction of Geotechnical Performance of Spudcan Foundations During Installation and Removal of Jack-Up Units*, InSafeJIP, EOG0574 Rev 1b, 18th November 2010
- [A.7.3-1] Det Norske Veritas, *Environmental Conditions and Environmental Loads*, DNV-RP-C205, Oslo, April 2009
- [A.7.3-2] Det Norske Veritas, *Strength Analysis of Main Structures of Self-Elevating Units*, DNV Classification Note 31.5, Oslo, February 1992
- [A.7.3-3] WHEELER, J.D., *Method for Calculating Forces Produced by Irregular Waves*, Proc. 1st Offshore Technology Conference, OTC 1006, Houston, May 1969
- [A.7.3-4] SARPKEYA, T. and ISAACSON, M., *Mechanics of Wave Forces on Offshore Structures*, Van Nostrand Reinhold Company, 1981
- [A.7.3-5] TAYLOR, P.H., *Current Blockage — Reduced Forces on Offshore Space-Frame Structures*, Proc. 23rd Offshore Technology Conference, OTC 6519, Houston, May 1991
- [A.7.3-6] American Petroleum Institute, *Recommended Practice for Planning, Designing and Constructing Fixed Offshore Platforms — Load and Resistance Factor Design*, API RP 2A-LRFD, First Edition, 1993 with 1997 Supplement
- [A.7.3-7] KIRBY, J.T. and CHEN, T.S., *Surface waves on vertically sheared flows: approximate dispersion relations*, J. Geophysical Research, Vol. 94, no. C1, pages 1013-1027, January 15 1989
- [A.8.3-1] Det Norske Veritas, *Strength Analysis of Main Structures of Self-Elevating Units*, DNV Classification Note 31.5, Oslo, February 1992
- [A.9.3-1] MARTIN, C., *Physical and numerical modelling of offshore foundations under combined loads*, DPhil Thesis, University of Oxford, 1994
- [A.9.3-2] HOSSAIN, M.S. and RANDOLPH, M.F., *New mechanism-based design approach for spudcan foundations on single layer clay*, J. Geotech. Geoenviron. Eng. (ASCE), Vol. 135, No. 9, pp. 1264-1274, 2009
- [A.9.3-3] SKEMPTON, A.W., *The Bearing Capacity of Clays*, Building Research Congress, 1951
- [A.9.3-4] YOUNG, A.G., REMMES, B. and MEYER, B., *Foundation Performance of Offshore Jack-Up Drilling Rigs*, Journal of Geotechnical Engineering, Vol. 110, No. 7, 1984
- [A.9.3-5] HOULSBY G.T. and MARTIN C.T., *Undrained Bearing Capacity Factors for Conical Footings on Clay*, Géotechnique, Vol. 53, No. 5, pp. 513-520, 2003
- [A.9.3-6] FINNIE, I.M.S. and RANDOLPH, J.F., *Punch-through and liquefaction induced failure of shallow foundations on calcareous sediments*, 7th International Conference on Behaviour of Offshore Structures BOSS '94, Massachusetts, pp. 217-230, 1994
- [A.9.3-7] ERBRICH, C.T., *Australian frontiers - spudcans on the edge*, Keynote lecture, in Gourvenec, S. and Cassidy, M. (Eds.), *ISFOG 2005: Proc. 1st Int. Symp. Frontiers in Offshore Geotechnics*, University of Western Australia, Perth, 19-21 September 2005, Taylor and Francis, London, pp. 49-74, 2005
- [A.9.3-8] MARTIN, C.M., *User guide for ABC - Analysis of Bearing Capacity*, Department of Engineering Science, University of Oxford, Report No. OUEL 2261/03, available from <http://www-civil.eng.ox.ac.uk/people/cmm/software/abc/>

- [A.9.3-9] POULOS, H.G. and CHUA, K.F., *Bearing capacity of foundations on calcareous sand*, Proc. 11th Int. Conf. on Soil Mechanics and Foundation Engineering, Vol. 3, pp. 1619-1622, 1985
- [A.9.3-10] LE TIRANT, P. and NAUROY, J.F., *Design guides for offshore structures: foundations in carbonate soils*, Editions Technip, Paris, 1994
- [A.9.3-11] FINNIE, I.M. and RANDOLPH, M.F., *Bearing response of shallow foundations in uncemented calcareous soil*, Proc. Int. Conf. Centrifuge '94, Singapore, pp. 535-540, 1994
- [A.9.3-12] DUTT, R.N. and INGRAM, W.B., *Bearing capacity of jack-up footings in carbonate granular sediments*, Proc. Int. Conf. on Engineering for Calcareous Sediments, Perth, Vol. 1, pp. 291-296, 1988
- [A.9.3-13] PAN, J., *The behaviour of shallow foundations on calcareous soil subjected to inclined load*, PhD Thesis, The University of Sydney, 1999
- [A.9.3-14] PAN, J., CARTER, J.P. and AIREY, D.W., *The influence of load inclination on the bearing capacity of a circular footing*, Proc. 8th Australia New Zealand Conf. on Geomechanics, Hobart, Vol. 1, pp. 299-305, 1999
- [A.9.3-15] BYRNE, B.W. and HOULSBY, G.T., *Observations of footing behaviour on loose carbonate sands*, Géotechnique, Vol. 51, No 5, pp. 463-466, 2001
- [A.9.3-16] YEUNG, S.K. and CARTER, J.P., *An Assessment of the Bearing Capacity of Calcareous and Silica Sands*, International Journal for Numerical and Analytical Methods in Geomechanics, Vol. 13, pp. 19-26, 1989
- [A.9.3-17] HOULSBY, G.T., EVANS, K.M. and SWEENEY, M., *End bearing capacity of model piles in layered carbonate soils*, Proc. Int. Conf. on Engineering for Calcareous Sediments, Perth, Australia, Balkema, Vol. 1, pp. 209-214, 1988
- [A.9.3-18] SHARP, D.E. and VAN SETERS, A.J., *Results and evaluation of plate load test results*, Proc. Int. Conf. on Engineering for Calcareous Sediments, Perth, Australia, Vol. 2. Balkema, pp. 473-483, 1988
- [A.9.3-19] RANDOLPH, M.F. and ERBRICH, C., *Design of shallow foundations for calcareous sediments*, Proc. 2nd Int. Conf. on Engineering for Calcareous Sediments, Bahrain, pp.361-378, 1999
- [A.9.3-20] ISLAM, M.K., *Constitutive models for carbonate sand and their application to footing problems*, PhD Thesis, University of Sydney, 1999
- [A.9.3-21] ISLAM, M.K., CARTER, J.P. and AIREY, D.W., *A study of surface footings on carbonate soils*, Proc. Darassulam, Brunei, 2001
- [A.9.3-22] RANDOLPH, M.F., FINNIE, I.M. and JOER, H., *Performance of shallow and deep foundations on calcareous soil*, Proc. Symp. on Foundations of Difficult Soils, Kagoshima, Japan, 1993
- [A.9.3-23] YAMAMOTO, N., RANDOLPH, M.F. and EINAV, I., *Simple formulas for the response of shallow foundations on compressible sands*. Int. J. for Geomechanics, ASCE 8:4 pp 230-230, 2008
- [A.9.3-24] YAMAMOTO, N., RANDOLPH, M.F. and EINAV, I., *A numerical study of the effect of foundation size for a wide range of sands*, Journal of Geotechnical and Geoenvironmental Engineering, ASCE., 135(1): pp. 37-45, 2009
- [A.9.3-25] BROWN, J.D. and MEYERHOF, G.G., *Experimental study of Bearing Capacity in Layered Soils*, Proc. 7th ICSMFE, Vol. 2, 1969
- [A.9.3-26] VESIC, A.S., *Bearing Capacity of Shallow Foundations*, Foundation Engineering Handbook (H.F. Winterkorn and H.Y. Fang, eds.), pp. 121-147, Van Nostrand, 1975

- [A.9.3-27] YOUNG, A.G., FOCHT, J.A. JR., *Subsurface Hazards Affect Mobile Jack-up Rig Operations*, Sounding, McClelland Engineers Inc., Houston, Texas, Vol. 3, No. 2, pp. 4-9, 1981
- [A.9.3-28] JACOBSEN, M., CHRISTENSEN, K.V., SORENSEN, C.S., *Gennemlokning of Tynde Sandlag (Penetration of Thin Sand Layers)*, Vag-och Vattenbyggaren 8-9, Sweden, 1977
- [A.9.3-29] HIGHAM, M.D., *Models of Jack-up Rig Foundations*, M.Sc. Thesis, University of Manchester, 1984
- [A.9.3-30] CRAIG, W.H. and CHUA, K., *Deep penetration of spudcan foundations on sand and clay*, Géotechnique, Vol. 40, No. 4, pp. 541-556, 1990
- [A.9.3-31] BAGLIONI, V.P., CHOW, G.S. and ENDLEY, S.N., *Jack-up Rig Foundation Stability in Stratified Soil Profiles*, Proc. 14th Offshore Technology Conference, OTC 4408, Houston, 1982
- [A.9.3-32] HANNA, A.M. and MEYERHOF, G.G., *Design Charts for Ultimate Bearing Capacity of Foundations on Sand Overlying Soft Clay*, Canadian Geotechnical Journal Vol. 17, 1980
- [A.9.3-33] TEH, K.L., LEUNG, C.F., CHOW, Y.K. and CASSIDY, M.J., *Centrifuge model study of spudcan penetration in sand overlying clay*, Géotechnique, Vol. 60, No. 11, pp. 825-842, 2010
- [A.9.3-34] DEAN, E.T.R., HSU, Y.S., JAMES, R.G., SCHOFIELD, A.N., MURFF, J.D. and WONG, P.C., *Centrifuge Modelling of 3-Leg Jackups with Non-Skirted and Skirted Spuds on Partially Drained Sand*, Proc. 27th Offshore Technology Conference, OTC 7839, Houston, 1995
- [A.9.3-35] CASSIDY, M.J., RANDOLPH, M.F. and BYRNE, B.W., *A plasticity model describing caisson behaviour in clay*, Applied Ocean Research, Volume 28, Issue 5, pp. 345-358, October 2006
- [A.9.3-36] WONG, P.C. and MURFF, J.D., *Dynamic Analysis of Jack-up Rigs Using Advanced Foundation Models*, Proc. OMAE, Houston, paper 94-1315, 1994
- [A.9.3-37] BAERHEIM, M., *Structural Effects of Foundation Fixity on a Large Jack-up*, Proc. 4th International Conference, The Jack-Up Platform, City University, London, 1993
- [A.9.3-38] VAN LANGEN, H. and HOSPERS, B., *Theoretical Model for Determining Rotational Behavior of Spudcans*, Proc. 25th Offshore Technology Conference, OTC 7302, Houston, 1993
- [A.9.3-39] TEMPLETON, J.S., LEWIS, D.R. and BREKKE, J.N., *Spudcan Fixity in Clay, First Findings of a 2003 IADC Study*, Proc. 9th International Conference, The Jack-Up Platform, City University, London, 2003
- [A.9.3-40] TEMPLETON, J.S., BREKKE, J.N. and LEWIS, D.R., *Spudcan Fixity in Clay, Final Findings of a Study for IADC*, Proc. 10th International Conference, The Jack-Up Platform, City University, London, 2005
- [A.9.3-41] TEMPLETON, J.S., *Jackup Foundation Performance in Clay*, Proc. 38th Offshore Technology Conference, OTC 18367, Houston, May 2006
- [A.9.3-42] MARTIN, C.M. and HOULSBY, G.T., *Combined loading of spudcan foundations on clay: laboratory tests*, Géotechnique, Vol. 50, No. 4 pp. 325-338, 2000
- [A.9.3-43] MARTIN, C.M. and HOULSBY, G.T., *Combined loading of spudcan foundations on clay: numerical modelling*, Géotechnique, Vol. 51 No 8 pp. 687-699, 2001
- [A.9.3-44] TEMPLETON, J.S., *Spudcan Fixity in Clay, Further Results from a Study for IADC*, Proc 12th International Conference, The Jack-Up Platform, City University, London, 2009
- [A.9.3-45] TAN, F.S.C., *Centrifuge and theoretical modelling of conical footings on sand*, Ph.D. Thesis, University of Cambridge, 1990

- [A.9.3-46] GOTTARDI, G. and BUTTERFIELD, R., *On the bearing capacity of surface footings on sand under general planar loads*, Soils and Foundations, Japanese Society of Soil Mech. and Fndn Engng, Vol. 33, No. 3, pp. 68-79, 1993
- [A.9.3-47] GOTTARDI, G. and BUTTERFIELD, R., *The displacements of a model rigid surface footing on dense sand under general planar loading*, Soils and Foundations, Japanese Society of Soil Mech. and Fndn Engng, Vol. 35, No. 3, pp. 71-82, 1995
- [A.9.3-48] GOTTARDI, G., HOULSBY, G.T. and BUTTERFIELD, R., *The Plastic Response of Circular Footings on Sand under General Planar Loading*, Géotechnique 49, No 4, pp. 453-470, 1999
- [A.9.3-49] BIENEN, B., BYRNE, B.W., HOULSBY, G.T. and CASSIDY, M.J., *Investigating six degrees of freedom loading of shallow foundations on sand*, Géotechnique Vol. 56, No. 6, pp. 367-379, 2006
- [A.9.3-50] CASSIDY, M.J., *Experimental observations of the combined loading behaviour of circular footings on loose silica sand*, Géotechnique, Vol. 57, No. 4, pp. 397-401, 2007
- [A.9.3-51] SVANØ, G., *Foundation Fixity Study for Jack-up Unit*, SINTEF report STF22 F96660, Trondheim, August 1996
- [A.9.3-52] HOULSBY, G.T. and CASSIDY, M.J., *A plasticity model for the behaviour of footings on sand under combined loading*, Géotechnique, Vol. 52, No. 2, pp. 117-129, 2002
- [A.9.3-53] CASSIDY, M.J., BYRNE, B.W. and HOULSBY, G.T., *Modelling the behaviour of circular footings under combined loading on loose carbonate sand*, Géotechnique, Vol. 52, No. 10, pp. 705-712, 2002
- [A.9.3-54] WINTERKORN, H.F. and FANG, H.Y., *Foundation Engineering Handbook*, Van Nostrand Reinhold Company, 1975
- [A.9.3-55] BELL, R.W., *The Analysis of Offshore Foundations Subjected to Combined Loading*, MSc. Thesis presented to the University of Oxford, 1991
- [A.9.3-56] ANDERSEN, K.H., *Cyclic clay data for foundation design of structures subjected to wave loading*, Invited lecture, Proc. International Conf. on Cyclic Behaviour of Soils and Liquefaction Phenomena, CBS04, Bochum, Germany, p. 371-387, 2004
- [A.9.3-57] TEMPLETON, J.S., III, *Spudcan Fixity in Clay, Findings from Additional Work in a Study for IADC*, Proc. 11th International Conference, The Jack-Up Platform, City University, London, 2007
- [A.9.3-58] VAN LANGEN, H., WONG, P.C. and DEAN, E.T.R., *Formulation and Validation of a Theoretical Model for Jack-up Foundation Load-Displacement Assessment*, Proc. 6th International Conference on the Jack-up Platform – Design, Construction and Operation, London, 1997
- [A.9.3-59] WONG, P.C., CHAO, J.C., MURFF, J.D., DEAN, E.T.R., JAMES, R.G., SCHOFIELD, A.N. and TSUKAMOTO, Y., *Jack-up Rig Foundation Modeling II*, Proc. 25th Offshore Technology Conference, Houston, OTC 7303, May 1993
- [A.9.3-60] CASSIDY, M.J., MARTIN, C.M. and HOULSBY, G.T. *Development and application of force resultant models describing jack-up foundation behaviour*, Marine Structures, Vol. 17, Issues 3-4, pp. 165-193, 2004
- [A.9.3-61] CASSIDY, M.J., HOULSBY, G.T., HOYLE, M.J.R. and MARCOM, M.A. *Determining appropriate stiffness levels for spudcan foundations using jack-up case records*, Proc. 21st Int. Conf. on Offshore Mechanics and Arctic Engineering (OMAE), Oslo, Norway, OMAE2002-28085, 2002
- [A.9.3-62] Noble Denton Europe and Oxford University, *The Calibration of SNAME Spudcan Footing Equations with Field Data*, Report No L19073/NDE/mjrh, Rev 5, London, November 2006, available from: <http://www.nodent.co.uk/iadc/fixity/>

- [A.9.3-63] ROESSET, J.M., *Stiffness and Damping Coefficients of Foundations*, ASCE SP Dynamic Response of Piled Foundations – Analytical Aspects, 1980
- [A.9.3-64] UESHITA, K. and MEYERHOF, G.G., *Surface Displacement of Multilayer Soil Systems*, J. Soil Mechanics and Foundations Division, ASCE Vol. 93, No. 5, 1967
- [A.9.3-65] BREKKE, J.N., MURFF, J.D. and LAMB, W.C., *Calibration of Jackup Leg Foundation Model Using Full-Scale Structural Measurements*, Proc. 21st Offshore Technology Conference, OTC 6127, Houston, 1989
- [A.9.3-66] MATLOCK, H., *Correlations for Design of Laterally Loaded Piles in Soft Clay*, Proc. 2nd Offshore Technology Conference, OTC 1204, Houston, 1970
- [A.9.4-1] Det Norske Veritas, *Classification Note 30.4 - Foundations*, February 1992
- [A.9.4-2] BRANSBY, M. and RANDOLPH, M., *Combined loading of skirted foundations*, Géotechnique, Vol. 48, No. 5, 1998
- [A.9.4-3] BRANSBY, M. and RANDOLPH, M., *The effect of embedment depth on the undrained response of skirted foundations to combined loading*, Soils and Foundations, Vol. 39, No. 4, 1999
- [A.9.4-4] BRANSBY, M. and YUN, G.-J., *The undrained capacity of skirted strip foundations under combined loading*, Géotechnique Vol. 59, No. 2, pp.115-125, 2009
- [A.9.4-5] CASSIDY, M.J., BYRNE, B.W. and RANDOLPH, M.F., *A comparison of the combined load behaviour of spudcan and caisson foundations on soft normally consolidated clay*, Géotechnique, Vol. 54, No. 2, pp. 91-106, 2004
- [A.9.4-6] EIDE, A., TUEN, K.A. and BAERHEIM, M., *The Yme Jack-up with Skirt Foundation*, Proc. 28th Offshore Technology Conference, OTC 8158, Houston, 1996
- [A.9.4-7] GOURVENEC, S., *Alternative design approach for skirted footings under general combined loading*, BGA International Conference on Foundations, Dundee, Thomas Telford, London, 2003
- [A.9.4-8] GOURVENEC, S., *Effect of embedment on the undrained capacity of shallow foundations under general loading*, Géotechnique, Vol. 58, No. 3, pp. 177–185, 2008
- [A.9.4-9] GOURVENEC, S. and RANDOLPH, M., *Effect of strength non-homogeneity on the bearing capacity of circular skirted foundations subjected to combined loading*, Proceedings of the 11th International Symposium on Offshore and Polar Engineering, Kyushu, Japan, 2002
- [A.9.4-10] KELLEZI, L., HOFSTEDE, H.W.L. and HANSEN, P.B., *Jack-up footing penetration and fixity analyses*, in Gourvenec, S. and Cassidy, M. (Eds.), *ISFOG 2005: Proc. 1st Int. Symp. Frontiers in Offshore Geotechnics*, University of Western Australia, Perth, 19-21 September 2005, Taylor and Francis, London, pp. 559-565, 2005
- [A.9.4-11] KELLEZI, L., KUDSK, G. and HOFSTEDE, H., *Skirted footings capacity for combined loads and layered soil conditions*, 2nd BGA International Conf. on Foundations, ICOF 2008, Dundee, Scotland, Proc. Volume 1 pp. 923-935, June 2008
- [A.9.4-12] KELLEZI, L., DENVER, H., KUDSK, G. and STADSGAARD, H., *FE skirted footings analyses for combined loads and layered soil profile*, Proc. 14th European Conf. on Soil Mech. and Geotech. Eng. ECSMGE, Madrid, Spain, September 2007
- [A.9.4-13] LELAND, A., SMEDVIG, A.S., MOMMAAS, C.J. and BAERHEIM, M., *Cost-Effective Spudcan Modification To Achieve Increased Bearing Capacity*, IADC/SPE Drilling Conference, Texas, 27443, 1994

- [A.9.4-14] SVANO, G. and TJELTA, T.I., *Skirted Spudcans — Extending Operational Depth and Improving Performance*, Marine Structures, Vol. 9, Issue 1, pp. 129-148, 1996
- [A.9.4-15] STEWART, D.P. and FINNIE, I.M.S., *Spudcan — Footprint Interaction During Jack-up Workovers*, Proceedings, 11th International Offshore and Polar Engineering Conference, Stavanger, Vol. 1, pp. 61-65, 2001
- [A.9.4-16] CASSIDY, M.J., QUAH, C.K. and FOO, K.S., *Experimental investigation of the reinstallation of spudcan footings close to existing footprints*. Journal of Geotechnical and Geoenvironmental Engineering, ASCE, Vol. 135, No. 4, pp. 474-486, 2009
- [A.9.4-17] DEAN, E.T.R. and SERRA, H., *Concepts for mitigation of spudcan-footprint interaction in normally consolidated clay*, Proc. 14th ISOPE Conf., Toulon, France, 2004
- [A.9.4-18] JARDINE, R.J., KOVECEVIC, N., HOYLE, M.J.R., SIDHU, H.K. and LETTY, A., *A study of eccentric jack-up penetration into infilled footprint craters*, Proc. 8th International Conference, The Jack-Up Platform, eds: C. D'Mello and L. Boswell, City University, London, 2001
- [A.9.4-19] TEH, K.L., BYRNE, B.W. and HOULSBY, G.T., *Effects of seabed irregularities on loads developed in legs of jack-up units*, Proc. of Jack-Up Asia Conference and Exhibition, PetroMin, Singapore, 7-8 December, 2006
- [A.9.4-20] GAUDIN, C., CASSIDY, M.J. and DONOVAN, T., *Spudcan reinstallation near existing footprints*, Proc. of the 6th International Offshore Site Investigation and Geotechnics Conference: Confronting New Challenges and Sharing Knowledge, Society of Underwater Technology, London, UK, pp. 285-292, 2007
- [A.9.4-21] GAN, C.T., LEUNG, C.F. and CHOW, Y.K., *A study on spudcan footprint interaction*, Proc. of the Second British Geotechnical Association International Conference on Foundations, ICOF 2008, Volume One, 2008
- [A.9.4-22] FOO, K., QUAH, M., WILDBERGER, P. and VAZQUEZ, J., *Spudcan Footprint Interaction and Rack Phase Difference*, Proc. 9th International Conference – The Jack-Up Platform, City University, 2003
- [A.9.4-23] HAMBLY, E.C., *Punch-through Instability of Jack-up on Seabed*, Journal of Geotechnical Engineering, ASCE, Vol. 111, No. 4, 1985
- [A.9.4-24] BYRNE, B.W. and CASSIDY, M.J., *Investigating the response of offshore foundations in soft clay soils*, Proc. OMAE'02 21st Int. Conf. On Offshore Mechanics and Arctic Engineering, Oslo, Norway, OMAE2002-28057, 2002
- [A.9.4-25] CRAIG, W.H., *Spudcan foundations: installation with deep penetration and subsequent removal*, Geotechnical Engineering, Vol. 131 No. 3, pp. 146-151, 1998
- [A.9.4-26] CRAIG, W.H. and CHUA, K., *Extraction forces for offshore foundations under undrained loading*, Journal of Geotechnical Engineering, Vol. 116, No. 5, pp. 868-884, 1990
- [A.9.4-27] CRAIG, W.H., CRUICKSHANK, M. and FISHER, R., *Seabed breakout forces*. Proc. Int. Conf. on Physical Modelling in Geotechnics 2002, Newfoundland, Canada. Balkema. pp. 283-287, 2002
- [A.9.4-28] ERBRICH, C.T., *Australian frontiers - spudcans on the edge*, Keynote lecture, in Gourvenec, S. and Cassidy, M. (Eds.), *ISFOG 2005: Proc. 1st Int. Symp. Frontiers in Offshore Geotechnics*, University of Western Australia, Perth, 19-21 September 2005, Taylor and Francis, London, pp. 49-74, 2005
- [A.9.4-29] PURWANA, O.A., LEUNG, C.F., CHOW, Y.K. and FOO, K.S., *Influence of base suction on extraction of jack-up spudcans*, Géotechnique 55(10):741-753, 2005

- [A.9.4-30] KRAMER, S.L., *Geotechnical Earthquake Engineering*, Prentice Hall, New Jersey, 1996
- [A.9.4-31] IDRIS, I.M. and BOULANGER, R.W., *Semi-empirical Procedures for Evaluating Liquefaction Potential During Earthquakes*, Proc. of the 11th International Conference on Soil Dynamics and Earthquake Engineering (ICSDEE) and the 3rd International Conference on Earthquake Geotechnical Engineering (ICEGE), Berkley, California, pp. 32-56, 2004
- [A.9.4-32] DEAN, E.T.R., *Some Potential Approximate Methods for the Preliminary Estimation of Excess Pore Water Pressures and Settlement-Time Curves for Submerged Foundations subjected to Time Dependent Loading*, Cambridge University Engineering Department, CUED/D-Soils/TR240, 1991
- [A.9.4-33] SWEENEY, M., WEBB, R.M. and WILKINSON, R.H., *Scour around jackup rig footings*, Proc. 20th Offshore Technology Conference, OTC 5764, Houston, 1988
- [A.9.4-34] WHITEHOUSE, R.J.S., *Scour at marine structures*, Thomas Telford Publications, London, 1998
- [A.9.4-35] RUDOLPH, D., ARNOUT, C., BOS, K.J. and RIETEMA, K., *Scour around spudcans – analysis of field measurements*, The Fifteenth International Offshore and Polar Engineering (ISOPE) Conference Korea, 2005
- [A.9.4-36] McDOWELL, D.M and O'CONNOR, B.A., *Hydraulic behaviour of estuaries*, Civil engineering hydraulics series, Macmillan, London, 1977
- [A.9.4-37] SICILIANO, R., HAMILTON, J., MURFF, J. and PHILLIPS, R., *Effect of jackup spud cans on piles*, Proc. 22nd Offshore Technology Conference, OTC6467, Houston, 1990
- [A.9.4-38] STEWART, D., *Influence of jack-up operation adjacent to a piled structure*, Proc. Int. Symposium on Frontiers in Offshore Geotechnics, Taylor and Francis, 2005
- [A.9.4-39] LEUNG, C., XIE, Y. and CHOW, Y.K., *Centrifuge Model Study of Spudcan-pile Interaction*, Proc. Sixteenth International Offshore and Polar Engineering Conference, San Francisco, California, USA, May 28-June 2, 2006
- [A.9.4-40] KELLEZI, L., KUDSK, G. and HANSEN, P.B., *FE Modelling of Spudcan – Pipeline Interaction*, in Gourvenec, S. and Cassidy, M. (Eds.), *ISFOG 2005: Proc. 1st Int. Symp. Frontiers in Offshore Geotechnics*, University of Western Australia, Perth, 19-21 September 2005, Taylor and Francis, London, pp. 551-557, 2005
- [A.9.4-41] US Department of the Interior Minerals Management Service, *Notice to Lessees and Operators of Federal Oil, Gas and Sulphur Leases and Pipeline Right of Way Holders in the Outer Shelf Gulf of Mexico OCS Region, Shallow Hazard Requirements*, NTL 2008-G05, April 1, 2008 (or its latest replacement)
- [A.9.4-42] OGP, *Geohazards from seafloor instability and mass flow*, The International Association of Oil and Gas Producers, London, Report No 425, December 2009
- [A.10.4-1] LYSMER, J. and RICHART, F. E., *Dynamic Response of Footings to Vertical Loading*, J. Soil Mec. and Found. Div., 92(SM1) 65-91, 1966
- [A.11.3-1] Det Norske Veritas, *Structural Design of Self Elevating Units, (LRFD Method)*, DNV-OS-C104, Oslo, October 2007
- [A.11.3-2] Det Norske Veritas, *Fatigue Design of Offshore Steel Structures*, DNV-RP-C203, April 2008
- [A.11.3-3] American Bureau of Shipping, *Guide for the Fatigue Assessment of Offshore Structures*, ABS 115, April 2003

- [A.11.3-4] American Bureau of Shipping, *Commentary on the Guide for the Fatigue Assessment of Offshore Structures*, ABS 115, April 2003
- [A.11.3-5] American Petroleum Institute, *Recommended Practice for Planning, Designing and Constructing Fixed Offshore Platforms — Load and Resistance Factor Design*, API RP 2A-LRFD, First Edition, 1993 with 1997 Supplement
- [A.11.3-6] Health and Safety Executive (HSE), *Steels*, OTO 2001:015, 2001
- [A.11.3-7] Health and Safety Executive (HSE), *Background to New Fatigue Guidance for Steel Joints and Connections in Offshore Structures*, OTH92 390, December 1999
- [A.12.5-1] American Institute for Steel Construction (AISC), *Specification for Structural Steel Buildings*, 13th Edition, ANSI/AISC 360-05, March 2005
- [A.12.5-2] British Standards Institute (BSI), *Steel, concrete and composite bridges. Code of practice for design of steel bridges*, BS 5400-3:2000, October 2000
- [A.12.5-3] Det Norske Veritas, *Buckling Strength Analysis of Bars and Frames, and Spherical Shells*, DNV Class Notes 30.1, Oslo, April 2004
- [A.12.5-4] American Petroleum Institute, *Stability Design of Cylindrical Shells*, API Bulletin 2U, June 2004
- [A.12.6-1] GALAMBOS, T.V., *Guide to stability design criteria for metal structures*, 5th Ed. J. Wiley and Son, 1998
- [A.12.6-2] DYER, A.P., *Plastic Strength Interaction Equations for Jack-Up Chords*, MSc Thesis, Dept of Mechanical Engineering, Univ. of Sheffield, Nov. 1992
- [A.12.6-3] EN 1993-1-1:2005 Eurocode 3: *Design of steel structures – Part 1-1: General rules and rules for buildings*
- [C.2-1] LU, Y., CHEN, Y.N., TAN, P.L., BAI, Y., *Prediction of Jack-Up Response* Proc. 20th Int. Conf. on Offshore Mechanics and Arctic Engineering (OMAE), Rio de Janeiro, Brazil, 2001
- [C.2-2] WINTERSTEIN, S.R., *Nonlinear Vibration Models for Extremes and Fatigue*, Journal of Engineering Mechanics, Vol. 114, No 10, 1988
- [C.2-3] JENSEN, J.J., *Dynamic Amplification of Offshore Steel Platform Responses due to Non-Gaussian Wave Loads*, The Danish Centre for Applied Mathematics and Mechanics, Report No 425, Submitted to Journal of Structural Engineering, ASCE, May 1991
- [C.2-4] Shell International Petroleum Maatschappij B.V., *Dynamic analysis and estimation of extreme response for jack-ups*, SIPM EPD/51/52, The Hague, August 1991
- [C.2-5] PERRY, M.J. and MOBBS, C.B., *Factoring to Improve Conservatism of the Drag-Inertia Method When Determining Dynamic Amplification of Jackup Units*, Proc. 30th Int. Conf. on Ocean, Offshore and Arctic Engineering (OMAE), Rotterdam, The Netherlands, OMAE2011-50335, 19-24 June 2011
- [D.1-1] RPS Energy, *Improved Guidelines for the Prediction of Geotechnical Performance of Spudcan Foundations During Installation and Removal of Jack-Up Units*, InSafeJIP, EOG0574 Rev 1b, 18th November 2010, Available from Internet: <<http://insafe.woking.rpsplc.co.uk/download.asp>>
- [E.2-1] CASSIDY, M.J. and HOULSBY, G.T., *Vertical bearing capacity factors for conical footings on sand*, Géotechnique, 52(9), pp. 687-692, 2002

- [E.2-2] American Petroleum Institute (API), *Recommended Practice 2A-WSD for Planning, Designing and Constructing Fixed Offshore Platforms—Working Stress Design*, API RP 2A-WSD, Twenty-first Edition, December 2000 with Errata and Supplement 3, Fig. 6.4.3.1, October 2007
- [E.2-3] PERKINS, S.W. and MADSON, C.R., *Bearing capacity of shallow foundations on sand: a relative density approach*, J. Geotechnical and Geoenvironmental Engineering, 126(6), pp. 521-530, 2000
- [E.2-4] RANDOLPH, M.F., JAMIOLKOWSKI, M.B. and ZDRAVKOVIC, L., *Load carrying capacity of foundations*, Proc. Skempton Memorial Conf., London, 1, 207-240, 2004
- [E.2-5] WHITE, D.J., TEH, K.L., LEUNG, C.F. and CHOW, Y.K., *A comparison of the bearing capacity of flat and conical circular foundations on sand*, Géotechnique, 58(10), pp. 771-779, 2008
- [E.2-6] SCHMERTMANN, J.H., HARTMANN, J.P. and BROWN, P.R., *Improved strain influence factor diagrams*, ASCE Journal of the Geotechnical Engineering Division, 104 (GT8), pp. 1131-1135, 1978
- [E.2-7] SCHMERTMANN, J.H., *Static Cone to Compute Static Settlement over Sand*, Journal of the Soil Mechanics and Foundation Division, ASCE, 96(SM3), 1970
- [E.2-8] SCHMERTMANN, J.H., *Guidelines for Cone Penetration Test Performance, and Design*. Federal Highway Administration Report, FHWA-TS-78-209, 1978
- [E.2-9] CASSIDY, M.J., HOULSBY, G.T., HOYLE, M.J.R. and MARCOM, M., *Determining appropriate stiffness levels for spudcan foundations using jack-up case records*, Proc. 21st Int. Conf. on Offshore Mechanics and Arctic Engineering, Oslo, paper 28085, 2002
- [E.3-1] HOSSAIN, M.F. and RANDOLPH, M.F., *New mechanism-based design approach for spudcan foundations on stiff-over soft-clay*, Proc. 41st Offshore Technology Conference, OTC19907, Houston, 2009
- [E.3-2] LEE, K.K., RANDOLPH, M.F. and CASSIDY, M.J., *New simplified conceptual model for spudcan foundations on sand overlying clay soils*, Proc. 41st Offshore Technology Conference, Houston. OTC20012, Houston, 2009
- [E.3-3] TEH, K.L., LEUNG, C.F., CHOW, Y.K. and HANDIDJAJA, P., *Prediction of punch-through for spudcan penetration in sand overlying clay*, Proc. 41st Offshore Technology Conference, OTC20060, Houston, 2009
- [E.3-4] LEE, K.K., *Investigation of potential spudcan punch-through failure on sand overlying clay soils*, PhD Thesis, The University of Western Australia, 2009
- [E.3-5] TEH, K.L., CASSIDY, M. J., LEUNG, C.F., CHOW, Y.K., RANDOLPH, M.F. and QUAH, C.K., *Revealing the bearing capacity mechanisms of a penetrating spudcan through sand overlying clay*, Géotechnique, Vol. 58, Issue 10, pp. 793-804, 2008
- [E.3-6] TEH, K.L., *Punch-through of spudcan foundation in sand overlying clay*, PhD Thesis, National University of Singapore, 2007
- [E.3-7] LOVE, J.P., BURD, H.J., MILLIGAN, W.E. and HOULSBY, G T., *Analytical and model studies of reinforcement of a layer of granular fill on a soft clay subgrade*, Can. Geotech. J. 24, pp. 611-622, 1987
- [E.3-8] BURD, H.J and FRYDMAN, S., *Bearing capacity of plane-strain footings on layered soils*, Can. Geotech. J. 34, pp. 241-253, 1997
- [E.3-9] CRAIG, W.H. and CHUA, K., *Deep penetration of spudcan foundations on sand and clay*, Géotechnique, Vol. 40, No. 4, pp. 541-556, 1990

- [E.3-10] TEH, K.L., LEUNG, C.F., CHOW, Y.K. and CASSIDY, M.J., *Centrifuge model study of spudcan penetration in sand overlying clay*, Géotechnique, Vol. 60, No. 11, pp. 825-842, 2010
- [F.2-1] DYER, A.P., *Plastic Strength Interaction Equations for Jack-Up Chords*, MSc Thesis, Dept of Mechanical Engineering, Univ. of Sheffield, Nov. 1992
- [F.2-2] DUAN, L. and CHEN, W.-F., *A Yield Surface Equation for Doubly Symmetrical Sections*, Engineering Structures, Vol. 12, pp. 114-119, April 1990
- [H.3-1] <http://www.boemre.gov> Bureau of ocean energy management, regulation and enforcement web site
- [H.3-2] STIFF, J.J., WILDBERGER, P., CARDONE, V.J., COX, A.T. and LEWIS, D.R., *Metocean Criteria for Jack-Ups in the Gulf of Mexico*, 38th Offshore Technology Conf., OTC 17879, May 2006

