

RECOMMENDED PRACTICE DNV-RP-C104

Self-elevating Units

JANUARY 2012

The electronic pdf version of this document found through <u>http://www.dnv.com</u> is the officially binding version

DET NORSKE VERITAS AS

FOREWORD

DET NORSKE VERITAS (DNV) is an autonomous and independent foundation with the objectives of safeguarding life, property and the environment, at sea and onshore. DNV undertakes classification, certification, and other verification and consultancy services relating to quality of ships, offshore units and installations, and onshore industries worldwide, and carries out research in relation to these functions.

DNV service documents consist of among others the following types of documents:

- Service Specifications. Procedual requirements.
- Standards. Technical requirements.
- Recommended Practices. Guidance.

The Standards and Recommended Practices are offered within the following areas:

A) Qualification, Quality and Safety Methodology

- B) Materials Technology
- C) Structures
- D) Systems
- E) Special Facilities
- F) Pipelines and Risers
- G) Asset Operation
- H) Marine Operations
- J) Cleaner Energy
- O) Subsea Systems

© Det Norske Veritas AS January 2012

Any comments may be sent by e-mail to rules@dnv.com

CHANGES

This document supersedes DNV-RP-C104, April 2011.

Text affected by the main changes in this edition is highlighted in red colour. However, where the changes involve a whole chapter, section or sub-section, only the title may be in red colour.

Main Changes January 2012

• General

This edition of the document contains only minor corrections and clarifications. Amended text is in red colour.

- Sec.2. Environmental Conditions
- Formula for wind velocity v(t,z) has been corrected.

CONTENTS

1.	The Self Elevating Unit	. 6
1.1	Introduction Important concept differences	6 6
1.3	Special features	0 6
1.4	General design principles	7
1.5	Load and Resistance Factored Design (LRFD)	/` 8
1.7	Abbreviations	8
2.	Environmental Conditions	. 8
2.1	Introduction	8
2.2	Waves	9 12
2.5	Wind	15
2.5	Water depth	15
2.6	Bottom conditions	16
3.	Design Loads	16
3.2	Permanent (G) and variable functional loads (O)	17
3.3	Deformation loads (D)	17
3.4	Environmental loads (E)	17
3.5 3.6	Accidental loads (A)	19
4.	Global Response Analysis	19
4.1	Introduction	19
4.2	Leg stiffness	19
4.3 4.4	Global analysis for the elevated condition	21 22
4.5	Global analysis for the transit condition	32
4.6	Global analysis for the installation and retrieval conditions	36
5.	Local structural analyses and considerations.	37
5.1 5.2	Spudcans and lower part of leg - Elevated condition	<i>31</i> 39
5.3	Leg joints by FEM analyses	43
5.4	Hull and jack house	43
6.	Structural strength (ULS)	45 45
6.2	Local strength of legs	43 47
6.3	Tubular joints	48
6.4 6.5	Hull (Deck) structure	51
6.6	Capacity for jacking and fixation systems	53
7.	Fatigue Strength (FLS)	53
7.1	Introduction	53
7.2	Stress concentration factors	54 54
7.4	S-N curves	54 54
7.5	Allowable extreme stress range	55
7.6	Different operating conditions	55
7.8	Simplified fatigue evaluation	56
7.9	Weibull parameter (h)	58
8.	Accidental strength (ALS)	60
8.1 8.2	General	60 60
8.3	Damaged structure	64
9.	Overturning stability analyses	65
9.1	Introduction	65
9.2 9 3	Stabilizing moment	65 66
9.4	Design requirement.	66

9.5	Foundation stability		
10.	Air gap		
10.1	Introduction		
10.2	Requirement		
10.3	Caution	67	
11.	References	68	
Арре	Appendix A. Simplified global leg model of lattice legs70		

1. The Self Elevating Unit

1.1 Introduction

This Recommended Practice (RP) presents recommendations for the strength analyses of main structures of self-elevating units.

The design principles, overall requirements, and guidelines for the structural design of self-elevating units are given in the DNV Offshore Standards:

- DNV-OS-C101 Design of Offshore Steel Structures, General (LRFD method), /1/.
- DNV-OS-C104 Structural Design of Self-elevating Units (LRFD method), /2/.
- DNV-OS-C201 Design of Offshore Units (WSD method), /3/.

The above standards refer to two safety formats:

- LRFD = Load and Resistance Factor Design method. See 1.5.
- WSD = Working Stress Design method. See 1.6.

The selected safety format must be followed for all components of the considered self-elevating unit, for example one can not use one safety format for the legs and another for the hull structure.

The units are normally designed to serve at least one of the following functions:

- production
- drilling
- accommodation
- special services (e.g. support vessel, windmill installation vessel, etc.).

1.2 Important concept differences

Modern jack-up platforms usually have three or four legs. The legs are normally vertical, but special designs with slightly tilted legs have been developed for better stability in the elevated condition. The legs are most commonly either designed as tubulars with circular or square cross section, or as lattice structures with triangular or square cross section.

A drilling slot may be cut into one side of the deck (typically the aft side), but for other platforms the derrick may be cantilevered over the side.

There are basically two different concepts for bottom support. Most jack-up platforms have separate legs with special footings (spud cans). Alternatively all legs are connected to a large mat, designed to prevent excessive penetration.

Some jack-ups are also supported by a pre-installed large tank structure resting on the sea-bed. The jack-up tubular legs/caisson may in such cases be connected to the bottom tank structure by grouted connections similar as have been used for support structures for offshore wind turbines. The grout may in such cases be important for the structural strength and behaviour of the complete platform.

Wind Turbine Installation units with compact tubular legs may be built without spud cans, i.e. the lower end of the leg is closed and represents the footprint on the seabed.

1.3 Special features

Different modes of operation or phases during the life of a self-elevating unit are usually characterised in terms of "design conditions". The following design conditions are normally to be considered:

- Installation
- Elevated (Operation and Survival)
- Retrieval
- Transit.

Design analyses tend to emphasize on the elevated condition, while statistics show that most accidents occur during transit, installation and retrieval.

A jack-up platform is normally designed with independent legs, and is therefore, with respect to global stiffness, rather flexible. The lateral stiffness is typically an order of magnitude less than the stiffness of a corresponding jacket structure. The important consequence of low stiffness is that dynamic effects should be taken into consideration, in particular for deeper waters and for areas with severe wave conditions.

Changes in the design conditions of a self-elevating unit are usually accompanied by significant changes in leg penetration, soil fixity, water depth, air gap, etc for the elevated condition.

Changes in draught, ballast, leg/spudcan submergence, etc will change design conditions for the transit condition.

A jack-up platform is a mobile unit, but it has narrow limits for operation. The designer will normally specify a limited range of environmental conditions for some of the design conditions.

These limitations must be clearly documented in the design analysis, in the operational manual and in the certificates of the platform. For example in DNV Appendix to Class Certificate.

It is the duty of the operator to carefully adhere to these limitations, so that they may also be applied in design.

However, in many cases the environmental and/or soil conditions on one specific location are more or less incomparable with the original assumptions. Effective methods for evaluation of an existing platform's suitability for a new location are therefore frequently needed.

1.4 General design principles

Structures and elements there of, shall possess ductile resistance unless the specified purpose requires otherwise.

Structural connections are, in general, to be designed with the aim to minimise stress concentrations and reduce complex stress flow patterns.

Structural strength shall be evaluated considering all relevant, realistic load conditions and combinations. Scantlings shall be determined on the basis of criteria that combine, in a rational manner, the effects of relevant global and local responses for each individual structural element.

Relevant load cases have to be established for the specific design conditions. The design is to be based on the most unfavourable combination. It is not always obvious which combination will be the worst for one specific part of the platform. It may therefore be necessary to investigate a number of load cases. Different load cases are obtained by different combinations of Permanent, Variable, Deformation and Environmental loads when referring to the LRFD format, /1/ and /2/. Functional, Environmental and Accidental loads are referred to in the WSD format, /3/

The design criteria for jack-up platforms relate to:

- Strength intact and damaged conditions (elevated and transit)
- Foundation and overturning stability (elevated)
- Air gap (elevated)
- Hydrostatic stability. (Compartmentation and stability requirements for intact and damaged condition in transit.)

Hydrostatic stability requirements are not further discussed in this classification note.

For design of the grout reference is made to DNV-OS-C502 Offshore Concrete Structures /6/ and DNV-OS-J101 Design of Offshore Wind Turbine Structures /7/. Research and experiments are ongoing pr November 2010, reference /6/ and /7/ are subject to further updates to include the latest up-to date "state of the art" for such grouted connections.

It is important to evaluate differences for the grouted connection for jack-up foundations vs. a wind turbine supports. Diameters, fatigue curves below or above water are quite different, possibility of cracks with subsequent washed out grout, etc.

The strength of grout/concrete will not be discussed in the present recommended practice.

Units intended to follow normal inspection intervals according to class requirements, i.e. typically drilling units with inspection in sheltered waters or dry dock every 5 years, shall be designed with the design fatigue life equal to the service life, minimum 20 years, as given in DNV-OS-C104 Sec.6 (LRFD) or DNV-OS-C201 Sec.7 and 12 (WSD).

Units intended to stay on location for prolonged period, i.e. typically production units without planned inspections in sheltered water or dry dock, shall also comply with the requirements given in DNV-OS-C104 Appendix A (LRFD) or DNV-OS-C201 Sec.12 and Appendix C (WSD). These supplementary requirements for permanently installed units are related to:

- site specific environmental criteria
- inspection and maintenance
- fatigue.

1.5 Load and Resistance Factored Design (LRFD)

Design by the LRFD method is a design method by which the target component safety level is obtained by applying load and resistance factors to characteristic reference values of loads (load effects) and structural resistance.

The general design principles with use of the LRFD method and different limit states are described in DNV-OS-C101 Sec.2. Design principles specific for self-elevating units are described in DNV-OS-C104 Sec.3.

A limit state is a condition beyond which a structure or part of a structure exceeds a specified design requirement.

A limit state formulation is used to express a design criterion in a mathematical form. The limit state function defines the boundary between fulfilment and contravention of the design criteria. This is usually expressed by an inequality, as in DNV-OS-C101 Sec.2 D201. The design requirement is fulfilled if the inequality is satisfied.

The design requirement is contravened if the inequality is not satisfied. The following limit states are included in the present RP:

- *Ultimate Limit States (ULS)* corresponding to the ultimate resistance for carrying loads
- Fatigue Limit States (FLS) related to the possibility of failure due to the effect of cyclic loading
- Accidental Limit States (ALS) corresponding to damage to components due to an accidental event or operational failure.

Table 1-1 indicates which limit states are usually considered in the various design conditions.

Table 1-1 Design conditions and limit states						
	Installation	Operating	Survival	Transit	Accidental	Damaged
ULS a)		x	x	x		
ULS b)	x	x	x	x		
FLS		x	x	x		
ALS					x	x

1.6 Working Stress Design (WSD)

In the WSD method the component safety level is obtained by checking the strength usage factors against permissible usage factors, i.e. load and resistance factors are not applied in WSD.

The design principles with use of the WSD method are described in DNV-OS-C201 Sec.2 for different design and loading conditions. DNV-OS-C201 Sec.12 describes the special consideration for self-elevating units by the WSD method, as for example relevant design conditions for jack-up platforms. Loading conditions in WSD are grouped as follows:

- a) Functional loads
- b) Maximum combination of environmental loads and associated functional loads
- c) Accidental loads and associated functional loads
- d) Annual most probable value of environmental loads and associated functional loads after credible failures, or after accidental events
- e) Annual most probable value of environmental loads and associated functional loads in a heeled condition corresponding to accidental flooding

1.7 Abbreviations

- ALS Accidental Limit States
- DAF Dynamic Amplification Factor
- DFF Design Fatigue Factor
- DNV Det Norske Veritas
- FEM Finite Element Method
- FLS Fatigue Limit States
- LRFD Load and Resistance Factor Design
- RAO Response Amplitude Operator
- RP Recommended Practice
- SCF Stress Concentration Factor ULS Ultimate Limit States
- ULS Ultimate Limit States
- WSD Working Stress Design

2. Environmental Conditions

2.1 Introduction

The suitability of a jack-up platform for a given location is normally governed by the environmental conditions on that location.

A jack-up platform may be designed for the specific environmental conditions of one location, or for one or more environmental conditions not necessarily related to any specific location.

The environmental conditions are described by a set of parameters for definition of:

- Waves
- Current
- Wind
- Temperature
- Water depth

— Bottom condition

— Snow and ice.

2.2 Waves

The most significant environmental loads for jack-up platforms are normally those induced by wave action. In order to establish the maximum response, the characteristics of waves have to be described in detail. The description of waves is related to the method chosen for the response analysis, see 4.4.

Deterministic methods are most frequently used in the design analysis of jack-up platforms. The sea state is then represented by regular waves defined by the parameters:

- Wave height, H
- Wave period, T.

The reference wave height for the elevated (survival) condition for a specific location is the 100 year wave, H_{100} , defined as the maximum wave with a return period equal to 100 years. For unrestricted service the 100 year wave may be taken as:

$$H_{100} = 32$$
 metres

There is no unique relation between wave height and wave period. However, an average relation is:

$$H = \left(\frac{T - 1, 0}{4, 1}\right)^{2, 5}$$

where H is in metres and T in seconds.

In order to ensure a sufficiently accurate calculation of the maximum response, it may be necessary to investigate a range of wave periods. However, it is normally not necessary to investigate periods longer than 18 seconds.

There is also a limitation of wave steepness. Wave steepness is defined by:

$$S = \frac{2\pi}{g} \frac{H}{T^2}$$

The wave steepness need not be taken greater than the 100 year wave steepness, which may be taken as /14/:

$$S_{100} = \frac{1}{7} \,, \, T \le 6$$

or

$$S_{100} = \frac{1}{7 + \frac{0.93}{H_{100}} (T^2 - 36)}, T > 6$$

where H_{100} is in metres and T in seconds. The relation between wave height and wave period according to these principles is shown in Figure 2-1.



Figure 2-1 Design wave height versus period.

Stochastic analysis methods are used when a representation of the irregular nature of the sea is required. A specific sea state is then described by a wave energy spectrum which is characterized by the following parameters:

- Significant wave height, H_s
- Average zero-up-crossing period, T_z

The probability of occurrence of a specific sea state defined by H_s and T_z is usually indicated in a wave scatter diagram, see DNV-RP-C205.

An appropriate type of wave spectrum should be used. However, unless the spectrum peak period is close to a

major peak in the response transfer function, e.g. resonance peak, the Pierson-Moskovitz spectrum may be assumed.

For fatigue analyses where long term effects are essential, the wave scatter diagram is divided into a finite number of sea states, each with a certain probability of occurrence.

For extreme response analysis, only sea states comprising waves of extreme height or extreme steepness need to be considered.

The most probable largest wave height in a specific sea state of a certain duration is:

$$H_{max} = H_s \sqrt{0.5 \ln N}$$

where N is the number of cycles in the sea state.

The duration of a storm is of the order of a few hours, and the number of cycles will normally be of order 10^3 . Consequently:

$$H_{max} \sim 1,85 H_s$$

The significant wave height need therefore normally not be taken greater than $0,55 H_{100}$. The steepness of a specific sea state is defined by:

$$S_s = \frac{2\pi}{g} \frac{H_s}{T_r^2}$$

The sea steepness need not be taken greater than the 100 year sea steepness for unrestricted service, which normally may be taken as /14/:

$$S_{100} = \frac{1}{10}, T_z \le 6$$

or

$$S_{100} = \frac{2}{15} - \frac{T_z}{180}, 6 < T_z < 12$$

or

The 100 year return period is used as the basis for extreme load analysis. For other types of analyses, different return periods may be used /1/.

 $S_{100} = \frac{1}{15}, 12 \le T_z$

In connection with fatigue analysis a return period equal to the required fatigue life is used as the basis for wave load analysis. The required fatigue life is normally 20 years /1/.

In connection with accidental loads or damaged conditions a return period of 1 year is taken as the basis for wave load analysis.

The maximum wave height corresponding to a specific return period may be obtained from a wave height exceedance diagram. If wave height exceedance data are plotted in a log/linear diagram, the resulting curve will in many cases be a straight line, see Figure 2-2. Such results are obtained for areas with a homogenous wave climate. Other results may be obtained for areas where the climate is characterized by long periods with calm weather interrupted by heavy storms of short duration /15/ and /16/.



When the individual waves have been defined, wave particle motions may be calculated by use of an appropriate wave theory, where shallow water effects and other limitations of the theory are to be duly considered, see e.g. /28/.

For deterministic response analysis, the following wave theories are generally recommended: Solitary wave theory:

$$\frac{\mathbf{h}}{\lambda} \leq 0,1$$

Stokes' 5th order wave theory:

$$0,1 < \frac{h}{\lambda} \le 0,3$$

Linear wave theory (or Stokes' 5th order):

$$0,3 < \frac{h}{\lambda}$$

where

h = still water depth.

 λ = wave length.

For stochastic response analysis, linear (Airy) wave theory are normally to be used for all applicable h/λ -ratios. When linear (Airy) wave theory is used, it is important that wave forces are calculated for the actual submerged portion of the legs.

In stochastic wave load analysis the effect of short-crested ness may be included by a directionality function, $f(\alpha)$, as follows:

$$\mathbf{S}(\omega, \alpha) = \mathbf{S}(\omega) \times \mathbf{f}(\alpha)$$

where

α = angle between direction of elementary wave trains and the main direction of short-crested wave system.

 $S(\omega, \alpha) = directional short-crested wave power density spectrum.$

= directionality function. $f(\alpha)$

In the absence of more reliable data the following directionality function may be applied:

$$f(\alpha) = C \cos^{n} \alpha \text{ for } -\frac{\pi}{2} \le \alpha \le \frac{\pi}{2}$$

where

= power constant. n

C = constant chosen such that

$$\int_{\alpha=-\pi/2}^{\alpha=\pi/2} f(\alpha) d\alpha = 1,0$$

The power constant, n, should normally not be taken less than:

= 4.0 for fatigue analysis when combined with Pierson-Moskowitz spectrum n

= 4.0 for extreme analysis. n

Calculation of wave crest elevation in connection with the air gap requirement, see 10.2, should always be based on a higher order wave theory.

2.3 Current

The current speed and profile are to be specified by the designer.

The current profile may in lieu of accurate field measurements be taken as (see Figure 2-3):

$$v_{\rm C} = v_{\rm T} \left(\frac{h-z}{h}\right)^{1/7} + v_{\rm W} \left(\frac{h_0-z}{h_0}\right)$$

where

- v_T = tidal current at still water level. v_W = wind generated current at still water level. h_o = reference depth for wind generated current (h_o = 50 m)
- z = distance from the still water level (positive downwards), but max. h_0
- h = still water depth.



Figure 2-3 **Current** profile

Although the tidal current velocity can be measured in the absence of waves, and the wind generated current velocity can be calculated, the resulting current velocity in the extreme storm condition is a rather uncertain quantity.

The wind generated current may be taken as:

$$v_{W} = 0.017 v_{R1}$$

where

 v_{R1} = wind velocity for z = 10 m/t = 1 min. See 2.4 z = height above still water

= averaging period t

It is normally assumed that waves and current are coincident in direction.

The variation in current profile with variation in water depth due to wave action is to be accounted for.

In such cases the current profile may be stretched or compressed vertically, but the current velocity at any proportion of the instantaneous depth is constant, see Figure 2-4. By this method the surface current component shall remain constant.



 $v_{c0} = v_{c1} = v_{c2}$ $A_{c1} > A_{c0} > A_{c2}$



2.4 Wind

The reference wind velocity, v_R , is defined as the wind velocity averaged over 10 minute, 10 m above the still water level.

The wind velocity as a function of height above the still water level and the averaging period may be taken as (see Figure 2-5):

$$v(t,z) = v_R \left(1 + 0.137 \ln \frac{z}{z_0} - 0.047 \ln \frac{t}{t_{10}} \right)$$

where

 v_R = Reference speed z = 10 m and t = 10 min

 z^{*} = height of load point above the still water level.

 z_o = reference height ($z_o = 10 \text{ m}$)

t = averaging time in minutes

 t_{10} = reference time = 10 minutes

It is normally assumed that wind, current and waves are coincident in direction.

1 minute averaging time (t = 1 minute in above equation) is used for sustained wind in combination with maximum wave forces.

For unrestricted operation the one minute wind speed need normally not be taken larger than v(1 min, 10 m) = 55 m/s.



Figure 2-5 Wind profile.

2.5 Water depth

The water depth is an important parameter in the calculation of wave and current loads. The required leg length depends primarily on the water depth, which therefore is a vital parameter for the evaluation of a jack-up's suitability for a given location.

Definitions:

The tidal range is defined as the range between the highest astronomical tide (HAT) and the lowest astronomical tide (LAT).

The mean water level (MWL) is defined as the mean level between the highest astronomical tide and the lowest astronomical tide.

The storm surge includes wind-induced and pressure-induced effects.

The still water level (SWL) is defined as the highest astronomical tide including storm surge.

The reference water depth (h) to be used for various calculations is the distance between the sea bed and the still water level (SWL), as defined in Figure 2-6.

STILL WATER LEVEL	(SWL)	
HIGHEST ASTRONOMICAL TIDE	(HAT)	STORM SURGE
MEAN WATER LEVEL	(MWL)	ASTRONOMICAL
LOWEST ASTRONOMICAL TIDE	(LAT)	TIDAL RANGE

Figure 2-6 Definition of water levels.

2.6 Bottom conditions

The bottom conditions have to be considered in the following contexts:

- The overturning stability depends on the stability of the foundation.
- The leg bending moments depend on the bottom restraint.
- The overall stiffness and consequently the natural period of the platform depends on the bottom restraint.
- The response at resonance depends on the damping which partly depends on the bottom conditions.
- The air gap depends on the penetration depth.

Requirements for verification of foundation behaviour during all phases of a jack-up platform at a specific location, including penetration, preloading, operation and pull-out are given in Classification Note No. 30.4 "Foundations" /10/.

A detailed treatment of bottom conditions can only be carried out in connection with one specific location. At the design stage, however, the detailed bottom conditions are normally not known. In such cases the boundary conditions for the leg at the seabed have to be established based on simplified and conservative assessments as indicated below.

The selected design values for the bottom conditions shall be stated in the certificates of the platform. At each new location it must be verified that the selected design values for the bottom conditions are met. The need for detailed analyses will in such cases depend on the degree to which the platform has previously been checked for similar conditions. When existing analyses are used as basis for verification of foundation behaviour, any deviation in actual conditions from those used in the analyses should be identified, and the uncertainties related to such deviations should be satisfactorily taken into account.

Legs with separate footings may penetrate the seabed to a considerable depth. The prediction of penetration depth may be vital when determining the suitability of a jack-up for a given location.

In certain conditions the spud-tanks may provide a considerable degree of rotational restraint for the leg, while for other conditions this moment restraint will be close to zero. These restoring moments at the seabed are very important because they have a direct effect on the following quantities:

- The leg bending moment distribution.
- The overall stiffness of the jack-up and consequently the lowest natural frequencies.
- The load distribution on the spud cans.

For simple structural analysis of jack-up platforms under extreme storm conditions, the leg/bottom interaction may normally be assumed to behave as pin joints, and thus unable to sustain any bending moments.

In cases where the inclusion of rotational seabed fixities are justified and included in the analysis, the model should also include lateral and vertical soil springs.

For further details see /12/.

For checking of spud cans, spudcan to leg connections and lower parts of the leg, a high bottom moment restraint should be assumed, see DNV-OS-C104 Ch.1 Sec.5 B200.

For fatigue analysis, bottom moment restraints may normally be included.

3. Design Loads

3.1 Introduction

The description used for loads in the current section mainly refers to LRFD-method, but the same loads will have to be designed for also when using the WSD-method.

As described in DNV-OS-C101 and DNV-OS-C104, the following load categories are relevant for selfelevating units:

- permanent loads (G)
- variable functional loads (Q)
- environmental loads (E)
- accidental loads (A)
- deformation loads (D).

Characteristic loads are reference values of loads to be used in the determination of load effects. The characteristic load is normally based upon a defined fractile in the upper end of the distribution function for the load. Note that the characteristic loads may differ for the different limit states and design conditions.

The basis for the selection of characteristic loads for the different load categories (G, Q, E, A, D), limit states (ULS, FLS, ALS) and design conditions are given in DNV-OS-C101 Sec.2 and 3.

A design load is obtained by multiplying the characteristic load by a load factor. A design load effect is the

most unfavourable combined load effect derived from design loads. Load factors are given in DNV-OS-C101 Sec.2.

3.2 Permanent (G) and variable functional loads (Q)

(i) Permanent loads (G)

Permanent loads are described/defined in DNV-OS-C101 Sec.3 C and DNV-OS-C104 Sec.3 B.

(ii) Variable functional loads (Q)

Variable functional loads are described/defined in DNV-OS-C101 Sec.3 D and DNV-OS-C104 Sec.3 C. This includes variable functional loads on deck area and tank pressures. In addition the deck load plan and tank plan specific for the considered unit need to be accounted for. Tank filling may vary between the design conditions.

(iii) Elevated hull weight (P + Q)

For a self-elevating unit it is normally limitations on the combinations of G and Q loads to be applied simultaneously on the hull. These limitations are normally expressed as maximum and minimum Elevated hull weight and an envelope for its horizontal position for centre of gravity (longitudinal and transverse directions).

3.3 Deformation loads (D)

Fabrication tolerances as out-of-straightness, hull leg clearances and heel of platform are to be considered. See 4.4.7 for description on how this can be included as $P-\Delta$ loads for the elevated condition.

3.4 Environmental loads (E)

(i) General

Environmental loads are in general terms given in DNV-OS-C101 Sec.3 E and F and in DNV-OS-C104 Sec.3 D. Practical information regarding environmental loads is given in the DNV-RP-C205.

(ii) Wave loads

Wave loads on jack-up legs may normally be calculated by use of the Morison equation. The force per unit length of a homogenous leg is then given by:

$$F = F_D + F_I$$

 $\begin{array}{l} F_{\rm D} &= 1/2 \ \rho \ C_{\rm D} \ D \ v \ |v| \ \text{-} \ \text{drag force} \\ F_{\rm I} &= \rho \ C_{\rm I} \ \text{a} \ \text{A} \ \text{-} \ \text{inertia} \ \text{force} \end{array}$

where:

 ρ = density of liquid.

- a = liquid particle acceleration.
- v = liquid particle velocity.
- A = cross sectional area of the leg (for a circular cylindrical leg, $A = \pi D^2 / 4$)
- D = cross sectional dimension perpendicular to the flow direction (for a circular cylindrical leg, D is the diameter).
- C_D = drag (shape) coefficient.
- C_{I} = inertia (mass) coefficient.

The liquid particle velocity and acceleration in regular waves are to be calculated according to recognized wave theories, taking into account the significance of shallow water and surface elevation, see 2.2. For a moving cylinder the equation has to be modified as indicated in 4.4.1.

Dynamic amplification of the wave loads is to be considered. This effect may be calculated based on 4.4.6.

(iii) Current loads

Current loads on jack-up legs may normally be calculated from the drag term in the Morison equation. The current velocity, as a function of depth below the still water level, may be determined in accordance with 2.3. Due to the non-linearity of drag forces it is not acceptable to calculate separately drag forces due to waves and current, and subsequently add the two linearly. In general the current velocity is to be added to the liquid particle velocity in the waves. The drag force is then calculated for the resulting velocity.

The maximum drag force due to the combined action of waves and current is approximately given by:

$$F_D = F_{DW} + 2\sqrt{F_{DW}F_{DC}} + F_{DC}$$

 F_{DW} = drag force due to waves. F_{DC} = drag force due to current. The mean value of the total drag force is approximately given by:

or

$$F_{DM} = 2\sqrt{R}F_{DW}$$
 if $F_{DW} > F_{DC}$
 $F_{DM} = (1 + R) F_{DW}$ if $F_{DW} < F_{DC}$

The amplitude of the total drag force is approximately given by:

 $F_{DA} = (1 + R) F_{DW}$ if $F_{DW} > F_{DC}$

or

$$F_{DA} = 2 \sqrt{R} F_{DW}$$
 if $F_{DW} < F_{DC}$

where

R = F_{DC} / F_{DW} (see Figure 3-1).



Figure 3-1 Drag force variation.

(iv) Wind loads

Wind loads are to be determined by relevant analytical methods and/or model test, as appropriate. Dynamic effects of wind are to be considered for structures or structural parts which are sensitive to dynamic wind loads. Wind forces and pressures on members above the sea surface may normally be considered as steady loads.

The steady state wind force or wind force component acting normal to the member axis or surface may be calculated according to:

$$F = 1/2 \rho C_S A v^2 \cos \alpha$$

where

 ρ = mass density of air (= 1.225 kg/m³ for dry air).

- C_s = shape coefficient for flow normal to the member axis.
- A° = projected area normal to the member axis.
- v = design wind velocity as defined in 2.4.
- α = angle between the direction of the wind and the cross sectional plane of the member.

The shape factor (C_s) is to be determined from relevant recognised data.

For building block methods the shape coefficients as given in Table 3-1 may be applied to the individual parts.

Examples of open lattice section are the drilling derrick and the part of the leg extending above the top of an enclosed jack house. Wind loads on the part of the leg between the wave crest and the hull baseline need normally not to be considered.

Table 3-1 Drag coefficients for building block method			
Part	Drag coeff.	Comments	
Deck side	1.0		
Deck houses	1.1	E.g. quarters, jack houses etc.	
Open lattice sections	2.0	Applied to 50% of the projected area	

For calculation of wind forces on individual beam members the wind load per unit length is given by:

$$F = \frac{1}{2} \rho C_D D v^2 \cos \beta$$

where

D = the characteristic cross-sectional dimension of the member.

 C_D = drag coefficient.

 β^{D} = angle between the direction of the wind and the cross sectional plane of the member.

The drag coefficient (C_D) is to be determined from relevant recognized data.

Solidification and shielding effects are to be taken into account if relevant.

For structures or part of structures sensitive to fluctuating wind forces, these forces are to be accounted for including dynamic effects. An example of structural parts prone to dynamic excitation by fluctuating wind forces are slender open lattice structures such as crane booms etc.

The possibility of local aerodynamic instability should be investigated, where relevant. Vortex shedding on slender members is such an example.

For general and more detailed information, reference is made to DNV-RP-C205 /8/.

3.5 Sea pressures during Transit (P, G and E loads)

Calculations of sea pressure acting on the bottom, side and weather deck of a self-elevating unit in transit condition may be done according to DNV-OS-C104 Sec.4 D600

3.6 Accidental loads (A)

Accidental loads are in general terms given in DNV-OS-C101 Sec.3 G and in DNV-OS-C104 Sec.3 D. Practical information regarding accidental loads is given in the DNV-RP-C204.

4. Global Response Analysis

4.1 Introduction

In the global response analysis it is determined how the various loads are distributed into the structure.

In the elevated condition the jack-up platform is comparable with a fixed jacket structure, but with more lateral flexibility due to the slenderness of the legs. The lateral flexibility is also pending on the moment restraint at the connection between the leg footing and the soil foundations. The jack-up will typically be subject to higher non-linear effects caused by large hull sway and more dynamic actions due to higher natural periods coinciding with or closer to the wave periods.

The elevated condition is normally critical for the major parts of the legs, spud cans and jack house and this condition may also be designing for the drill floor, the cantilever and sometimes the main barge girders in way of the legs supports. Also this condition may be critical for some internal bulkheads, in particular preload tanks which are not used in transit condition.

The transit condition is critical for the lower part of the legs and possibly also the jack house.

The transit condition may also be critical for the major part of the barge, due to hydrostatic loading and large motion induced leg bending moments.

For both conditions different analysis methods have been established. The choice of analysis method depends on the actual requirement for accuracy.

The elevated condition is normally critical for the major parts of the legs, spud cans and jack house while the transit condition is critical for the lower part of the legs and possibly also the jack house.

For the major part of the barge, the transit condition will often be critical due to hydrostatic loading and large motion induced leg bending moments. The elevated condition, however, may be critical for some internal bulkheads, in particular preload tanks which are not used in transit, drill floor and cantilever and sometimes the main barge girders in way of the legs supports.

In the static elevated condition, the barge may sometimes be assumed simply supported at the leg positions as the legs are clamped after elevation or the pinions are moving with different speed during elevation to account for the barge deflection (sagging). For jack-ups platforms with small guide clearances, guide/leg contact and friction may occur during jacking, giving some clamping moment, which should be included in the response analysis.

It may be the case that the leg experience significant bending restriction at the connection to the seabed. Such design condition is to be considered, by varying the leg/soil interaction as necessary within the design specifications to provide maximum stress in spud can and the lower end of the legs.

4.2 Leg stiffness

The leg stiffness has to be determined for the global response analysis. In particular the leg stiffness is essential for the calculation of second order bending effects and dynamic structural response. The Euler load of legs is described in Appendix A.2.

The leg structure may be grouped as follows:

- Tubular or box shaped legs
- Lattice legs
- Equivalent legs

The typical leg sections are illustrated in Figure 4-1.

4.2.1 Effect of rack teeth

The leg stiffness used in the overall response analysis may account for a contribution from a portion of the rack tooth material. The assumed effective area of the rack teeth should not exceed 10% of their maximum cross sectional area. When checking the capacity of the chords no account of rack teeth is to be considered. See Figure 4-2.



Figure 4-1 Typical leg sections.



W = Width of effective area (shaded) TH = Rack teeth height

Figure 4-2

Effective stiffness for chord including rack teeth

4.2.2 Stiffness of Lattice legs

The stiffness of a lattice leg may be determined either from a direct analysis of the complete structure by use of an appropriate computer program, or from a simplified analysis of an equivalent leg.

In the direct analysis each chord, brace and span-breaker member is represented in a detailed leg model.

The Stiffness of equivalent legs can be calculated from Appendix A.

4.2.3 Stiffness of tubular or box shaped legs

The stiffness of a cylindrical or box shaped jack-up legs are characterized by the beam properties.

A = cross sectional area.

 A_Q = shear area.

I = moment of inertia.

 I_{T} = torsional moment of inertia.

4.3 Leg-to-hull interaction

4.3.1 Basis for modelling the leg-to-hull connection

The most highly loaded part of a jack-up leg is normally just below/at the lower guide for the elevated condition (for a unit without a chocking system) and just above/at the upper guide for the transit condition. The leg/hull interaction depends very much on the actual design concept.

(i) Fixation system

The fixation system (also referred to as clamping system or rack chock system) is a piece of equipment with racks, mounted rigidly on the hull, which is engaged (grips the chord racks firmly) when relative motion between leg and hull is not required, i.e. when the hull is jacked to the desired operational position and during transit from one location to another. The purpose of the fixation system is to transfer the bending moment between leg and hull by vertical tension and compression forces on opposite leg chords.

For units with a fixation system, the jacking system is normally not engaged with the leg when the hull is in the elevated condition. At the initial position, the leg connects to the hull structure through the fixation system and the leg chord does not have any contact with the guide structure. After the leg is bent due to the imposed environmental loads, depending upon the gap between the leg chord and guide structure, the leg chord will contact with the guide structure. Therefore, the leg bending moment is resisted by the fixation system and guide structure.

(ii) Jacking system (pinions)

- A fixed jacking system is one which is rigidly mounted to the jack house and hence to the hull
- A floating jacking system is one which is mounted to the jack house via flexible upper and lower shock
 pads. Under environmental loading such a flexible system rotates and the guides come into contact with
 legs and resist a considerable proportion of the leg bending moment.

(iii) Upper and lower guides

The upper and lower guides will mainly transfer horizontal forces between the legs and the hull structure.

The forces transferred are dependent on the guide arrangement, and with one of the following alternative:

- Horizontal force only in the plane of the rack is transferred.
- Horizontal forces in two horizontal directions (orthogonal)

The guides take part in transfer of the horizontal forces and moments from the legs to the hull structure. Parts of this load transfer are also taken by the fixation system and the jacking system when they are engaged.

Tolerances and/or friction between the guides (part of hull) and the leg may have to be considered.

(iv) Leg-to-hull interaction for a detailed lattice leg model

For analysis of the global response with a detailed leg model, the leg-to-hull connections may typically be represented by short beams as shown on Figure 4-3.

These beams may be modelled by "hinged" connections at the ends closest to the hull/jacking structure. The hinges are modelled such that displacements and/or rotations are released to secure that forces and/or moments are only transferred in the relevant degrees of freedom. Normally it is sufficient accurate to make the short beams relative stiff. But the stiffness properties of the beams may also be adjusted to represent flexible leg-to-hull connections, for example if the jacking pinions are mounted on flexible pads. The leg-to-hull connections are:

- LG = Lower Guide
- FIX = Fixation system
- PIN = Jacking Pinions
- UG = Upper Guide





Typical leg-to-hull connection detailed leg model. Hull and jack house simplified with beams

4.3.2 Leg-to-hull for equivalent beam model of legs

See Appendix A.

4.4 Global analysis for the elevated condition

4.4.1 Introduction

The response analysis for the elevated condition should account for:

Dynamic response: The fundamental mode of large jack-up platforms corresponds to a natural period of the order of 5 seconds. This means that dynamic effects are significant and have to be accounted for.

Stochastic response: The most significant environmental loads are those induced by wave action. The irregularity of sea can only be simulated by use of a stochastic wave model.

Non-linear response: Equations governing the response of a jack-up platform are non-linear for several reasons:

- The selected wave theory may be non-linear.
- The drag term in the Morison equation is non-linear.
- Current loading interacts with wave loading and introduces non-zero mean loading.
- The submerged portion of the legs is variable.
- Second-order bending due to axial loading reduces the effective lateral stiffness.
- The interaction leg/hull and the interaction leg/sea bottom are non-linear.

However, a number of options for simplification of the analysis are indicated in Figure 4-4.



Figure 4-4 Methods for response analysis

The main considerations are:

- Dynamic versus static analysis
- Stochastic versus deterministic analysis

— Non-linear versus linear analysis.

Comparison studies between the various methods may be found in /11/, /23/. The dynamic equation of equilibrium is considered for illustration:

$$mr + cr + kr = F$$

$$F = c_d(v-r) |v-r| + c_f a + c_m(a-r)$$

where,

m = mass.

- c = damping, c = c (r).
- k = stiffness, k = k (r).
- r = displacement of structure.
- r = velocity of structure.
- r = acceleration of structure.
- a = acceleration of fluid.
- v = velocity of fluid.
- $c_d = drag$ force coefficient.
- c_{f}^{u} = Froude-Kryloff force coefficient.
- $c_m = added mass.$

4.4.2 Analysis methods

The analysis methods A to F from Figure 4-4 are described below.

(i) Method A. Stochastic Non-Linear Dynamic Analysis

Method A is the most comprehensive of the methods. In principle it is possible to account for all of the special effects mentioned above. However, the method requires long computer times and preliminary calculations with simplified methods should be conducted in advance. The equations of motion are as indicated in 4.4.1 generalized to cover the required motion components.

The waves may be simulated by the velocity potential on the form:

$$\Phi(\mathbf{x}, \mathbf{z}, \mathbf{t}) = \sum_{i=1}^{M} A_i \frac{\omega_i}{k_i} \frac{\cosh k_i (\mathbf{z} + \mathbf{h})}{\sinh k\mathbf{h}} \cos(\omega_i \mathbf{t} - k_i \mathbf{x} - \phi_i)$$

possibly extended to more wave directions, where

$\Phi(\mathbf{x},\mathbf{z},\mathbf{t})$	= velocity potential at location x , z and time t.
Ai	= amplitude of partial wave number i
	$=\sqrt{2 \mathrm{S}(\omega_{\mathrm{i}}) \Delta \omega_{\mathrm{i}}}$
h	= water depth.
$\omega_{\rm i}$	= angular frequency of wave number i.
k _i	= wave number connected to ω_i through the dispersion relationship.
$\dot{\phi_{\mathrm{I}}}$	= random phase, uniformly distributed between 0 and 2 π .
$\Delta \omega_{\rm i}$	= frequency band width associated with ω_i , i.e. $(\omega_{i+1} - \omega_i)$
$S(\omega)$	= wave spectrum.

Recommended values for some important variables are:

- Highest frequency: $\omega_{\rm M} = 2.5 \times 2\pi/T_z$ where T_z is average wave period.
- Sampling period of the wave record: $\Delta t = 0.5 \times 2\pi/\omega_{\rm M}$ usually about 1 to 2 seconds.
- Duration of simulated wave record: $T_D = 2048 \Delta t$. However, not less than 40 minutes for a single simulation.

The frequencies ω_i may be chosen with fixed frequency intervals equal to $\Delta \omega = 2 \pi / T_D$ whereby Fast Fourier Transform techniques may be applied in the summation of partial waves.

Alternatively the frequencies may be selected with irregular intervals $\Delta \omega_{\rm l}$. Number M of partial waves may then be from 25 to 50. In the region about a resonance frequency $\Delta \omega_{\rm l}$ with damping ratio $\xi_{\rm r}$, the band widths should satisfy $\Delta \omega_{\rm l} < 1/4 \pi \xi_{\rm r} \omega_{\rm r}$

Velocities and acceleration components are derived from the velocity potential in the nodes where forces are to be calculated.

The equations of motion may then be solved in the time domain by recognized methods as for instance the Newmark- β method. Recommended time step in the time integration is 1/20 of the resonance period $2\pi/\omega_r$ of the highest vibration mode.

Short transients from the time histories should be removed, e.g. the first 200 time steps are deleted from each time history in the statistical post processing.

Wave velocity and acceleration in the time steps that are not covered by the actual simulation may be found by interpolation. The interpolation method should avoid discontinuities. The method with cubic splines based on 4 to 6 of the simulated points may be recommended.

Dynamic equilibrium at each time step of the integration may be established by the Newton-Raphson iteration procedure.

Extreme response within the storm is then found by utilization of an acceptable "most probable maxima" extrapolation technique.

(Note: It is important that a seastate pre-qualification is undertaken prior to acceptance of such simulated seastate being accepted for use in the stochastic analysis.)

(ii) Method B. Deterministic Non-Linear Dynamic Analysis

Method B is similar to method A except that only regular waves are considered. Fluid velocity and acceleration are determined from the most accurate wave theory, and the non-linear equations of equilibrium are solved by time integration. The method is well suited for extreme response analysis, but not for rigorous fatigue analysis.

(iii) Method C. Stochastic Linear Dynamic Analysis

Method C is based on a linearization of the equation of equilibrium which may then be rearranged as:

$$(m + c_m) \ddot{r} + (c_l + c_{dl}) \dot{r} + k_l r = F_l$$

where

 $\begin{array}{ll} F_1 &= c_{dl} + c_i \ a \\ c_i &= c_f + c_m \\ c_{dl} &= c_d \ (v - \dot{r})_{ref} - \ Alternative \ 1. \\ &= c_d \sqrt{(8/\pi)} \ \sigma_{v-r} - \ Alternative \ 2. \end{array}$

 c_l, k_l = linearized damping and stiffness.

Two methods which are frequently used for linearization of the drag term are shown above, alternative 1 and 2. In alternative 1 the linearization is based on a reference relative velocity, $(v - r)_{ref}$, evaluated for waves of a predetermined steepness. The reference steepness for extreme load analysis may be frequency dependent as shown in 2.2. For fatigue analysis the reference velocity should be smaller than for extreme load analysis.

In alternative 2 the linearization is based on the standard deviation of the relative velocity which implies that an iterative procedure is required for the evaluation of the spectral density of the response. For practical purposes the standard deviation of the relative velocity is determined for each component as follows:

$$\sigma_{(v-\dot{r})} = \sqrt{\sigma_v^2 + \sigma_{\dot{r}}^2 - 2\sigma_{\dot{r}v}}$$

where σ_v and σ_r represent standard deviation for v and r respectively. This method is suited for extreme load analysis. Alternative 1 has the most straightforward physical interpretation, and by this method it is also possible to calculate the drag force for finite wave heights and thus account for the effect of a variable submerged volume.

The linearized equation of equilibrium is solved in the frequency domain by use of the standard method for linear stochastic analysis. This method rests on the assumption that the response spectrum in question can be represented by the product of a transfer function of the response squared and the wave spectrum. The most probable largest response is computed on the basis of information inherent in the response spectrum. The procedure may be used in combination with the normal mode approach which in the case of a jack-up platform may be very cost efficient. However, there are also obvious disadvantages of the method:

Linearization of drag forces introduces uncertainties.

The estimation of total damping is uncertain.

The effect of current cannot be included consistently.

Extrapolation to extremes is uncertain, especially due to the linearization.

Instantaneous load distributions are difficult to obtain.

The method is considered as a suitable approach for fatigue analysis, where the randomness of the sea is essential.

(iv) Method D. Deterministic Non-Linear Static Analysis

Method D is equivalent to method B for very stiff platforms, for which dynamic effects are insignificant. The analysis is considerably simplified because the equation of equilibrium is reduced to:

$$k r = F_s$$

where $F_s = c_d v |v| + c_i a$

However, jack-up platforms are in general so flexible that dynamic effects should not be neglected unless the effect is compensated by other conservative assumptions.

Simplified procedures which account for dynamic effects are given in 4.4.3 and onwards.

(v) Method E. Stochastic Linear Static Analysis

Method E is equivalent to method C for very stiff platforms, for which dynamic effects are insignificant.

(vi) Method F. Deterministic Linear Static Analysis

Method F is the most simple of all methods, and in general a number of important effects are ignored.

However, as discussed in connection with the other methods it is often possible to account for special effects by simple modifications. In many cases method F may be modified in such a way that the accuracy is not significantly reduced in comparison with method B. The main corrections will contain:

Dynamic amplification of the wave/current load can be accounted for by a horizontal "inertia" load in hull centre. See 4.4.3 to 4.4.6 for calculation methods.

P- Δ effect is included to account for the increased moments at the upper parts of the legs, including the non-linear amplification factor α . The non-linear effect of large hull displacement can be accounted for by a horizontal load in hull centre, see 4.4.7 for calculation methods.

The main advantage of the method is that it is very easy to establish instantaneous load distributions, and it is possible to work with very large and detailed structural models. See Figure 4-5 for typical global model.



Figure 4-5 Typical global model for Method F (Quasi-static)

(vii) Method A-F conclusions

A rigorous analysis corresponding to method A is possible for special investigations, but expensive. Deterministic methods may be used for extreme response analysis. Dynamic effects and non-linear effects should be accounted for, but this may be done by approximate modifications of a linear/static analysis. Stochastic methods should be used for fatigue analysis. However, it may be sufficient only to carry out a simplified fatigue analysis according to 7.8. In that case a deterministic response analysis is sufficient.

Further details may be found in /11/, /13/

4.4.3 Natural Period for the elevated condition

The natural period T_o may normally be found from an eigenvalue analysis of the global model.

The global model should represent the stiffness properties of the legs correctly. For braced legs, the global model may include each chord-, brace- and span breaker modelled as beams.

The bottom of the leg may be simplified modelled to represent the relevant soil condition (hinged or spoil springs) and the hull may be simplified modelled as beam grid with vertical beams for the jacking structure. The stiffness and load transfer at the leg to hull connection should be modelled to represents actual load transfer for the upper/lower guides, fixation system and jacking pinions.

See Figure 4-5 for a typical global analysis model.

The total mass is included by the modelled elevated hull weight, legs and spudcan weights. Added mass of submerged leg members and water filled legs should also be accounted for. Normally it is the elevated hull weight that is dominating and is most important for the natural period for a typical jack-up.

For simplified equivalent leg models of a jack-up, a procedure to calculate T_o is given in Appendix A.4.

For the elevated condition it is normally found that the three lowest natural frequencies correspond to the following motions:

- Longitudinal displacement (surge)
- Transverse displacement (sway)
- Torsional rotation (yaw).

4.4.4 Dynamic Amplification Factor (DAF)

Amplification of stresses due to dynamic structural response is taken into account in the ultimate strength and the fatigue strength evaluations.

The effect of dynamic amplification may be significant for the evaluation of fatigue strength, because the frequently occurring waves may have frequencies corresponding to the lowest natural frequencies of the structure.

Therefore dynamic effects due to environmental loading (wave/current) are to be accounted for in the response analysis.

A dynamic analysis may be required if the dynamic effects may be significant, resulting in dynamic amplification factors DAF > 1.1. Quasi-static analysis may, however, be used provided the dynamic effects are accounted for by an acceptable procedure. Examples of the latter are given in 4.4.6, where the dynamic amplification is accounted for by scaling horizontal load(s) applied in the hull centre.

(i) Simplified dynamic amplification factor (DAF) for ULS

This method is based on the assumption that the dynamic characteristics of a jack-up platform may be accounted for by use of a dynamic amplification factor (DAF), for a single degree of freedom system:

$$DAF = \frac{1}{\sqrt{\left[1 - \left(\frac{To}{T}\right)^2\right]^2 + \left(2\xi\frac{To}{T}\right)^2}}$$

where

To = natural period, see 4.4.3.

T = period of variable load (wave period).

 ξ = damping ratio, see 4.4.5.

(ii) Stochastic dynamic amplification factor (DAF_8) for ULS

The dynamic effects from wave loads may be assessed based on "stochastic" calculations of the significant response as obtained both from a dynamic analysis and from a quasi-static analysis. The displacement in the hull centre in the considered wave direction may typically be chosen as the response variable (Transfer function) used to determine the dynamic amplification factor (DAF_S) :

$$DAF_s = \frac{\Delta_{sig,Dynamic}}{\Delta_{sig,Static}}$$

Where:

 $\Delta_{sig,Dynamic}$ = Significant hull displacement (amplitude) from a dynamic forced response analysis

$$\Delta_{sig,dynamic} = 2\int_{0}^{\infty} H_{dyn}^{2}(\omega) \cdot S(\omega) d\omega$$

 $\Delta_{\text{sig static}} = \text{Significant hull displacement (amplitude) from a quasi-static analysis}$

$$\Delta_{sig,static} = 2 \int_{0}^{\infty} H_{stat}^{2}(\omega) \cdot S(\omega) d\omega$$

 $H_{dyn}(\omega), H_{stat}(\omega) =$ Transfer function (hull displacement) for dynamic and static analyses, respectively. S(ω) = sea spectrum (same for static and dynamic analyses

The dynamic forced response analysis and the quasi-static analysis are both to be based on the same wave frequencies (ω).

To represent the transfer function $H(\omega)$ with sufficient accuracy, the wave load frequencies (ω) in the analyses are to be selected carefully around natural periods for the platform and around periods with wave load cancellation/amplification.

The wave load calculation is normally sufficient if based on Airy 1st order linear wave theory.

Further ω should be selected over a sufficient range to cover the wave frequencies for the transfer functions and sea spectra applied. ω will normally correspond to wave periods in the range 2-30 sec for a typical jack-up.

A number of sea spectra $S(\omega)$ should be selected to cover the governing dynamic case, i.e. sufficient combinations of H_s/T_z -pairs to cover the wave climate.

(iii) Dynamic effects for fatigue (FLS)

The dynamic response will normally be included directly in a stochastic forced response analysis (frequency domain). Wave load frequencies are selected as described in (ii) above and damping as described in 4.4.5. The probabilities of the different sea states (H_s/T_z -pairs) are by the representative scatter diagram(s).

Thus the DAF-methods (i) and (ii) above is typically not used in fatigue to include dynamic amplification, except for when simplified fatigue calculations are carried out according to 7.8.

4.4.5 Damping

The damping ratio, ξ , to be used in the evaluation of DAF is the modal damping ratio. By definition:

$$\xi = \frac{c}{c_{cr}} = \frac{c}{2\sqrt{m\,k}}$$

where m, c and k are coefficients of mass, damping and spring in the equivalent one-degree-of-freedom system.

This is a quantity which depends on a number of variables. It should be observed that ξ increases with decreasing stiffness. This is important because the stiffness of jack-up platforms may be an order of magnitude less than the stiffness of a corresponding jacket.

The total damping coefficient, c, includes structural damping, hydrodynamic damping and soil damping. All of these contributions are difficult to determine, and none of them should be neglected.

The structural damping includes damping in guides, shock pads, locking devices and jacking mechanisms which means that the structural damping is design dependent. Typical structural damping is expected to be 1-3%.

The soil damping depends on spud can design and bottom conditions which means that the soil damping will be design dependent and site dependent. Typical soil damping is expected to be 0-2%.

The hydrodynamic damping depends on leg structure, drag coefficients and relative water particle velocity. This means that the hydrodynamic damping is not only design dependent, but it also depends on sea conditions and marine growth. Typical hydrodynamic damping is expected to be 2-4%.

It is thus obvious that it is very difficult to select a representative total damping ratio /15/ and /20/.

The damping should be based on measured values from the actual platform, or from platforms with similar spudcans and jacking systems.

If measurements do not exist a damping of 3-6% can be expected for FLS analysis.

For the storm analysis the damping is expected to be 6-9% and should not be taken higher than 7% without further justification.

As described in 2.2 it may be necessary to investigate a range of wave periods in order to determine the extreme response. If this range comprises the natural period of the platform, the simple deterministic approach where dynamic effects are taken into account by the DAF method (i) in 4.4.4, may be unreasonably conservative. In such cases it is suggested to use DAF method (ii) in 4.4.4.

The above is based on that the shape of the transfer function is characterized by a sharp peak at the natural period. This peak is narrow compared to the width of a realistic wave energy spectrum. Only a fraction of the wave energy will then correspond to this peak. In a deterministic approach the total wave energy is concentrated at one specific wave period, and if this period is taken at or close to the natural period, the response will be very severe (resonance).

Simplified calculation method for of damping at resonance is included Appendix A5.

4.4.6 Inertia loads (ULS)

The Inertia load (F_I) described below is meant to account for the dynamic amplification of the wave loads acting in the elevated condition.

The calculated dynamic amplification factor (DAF or DAF_S) in 4.4.4 may be used in combination with the approach below to determine the inertia load F_I . The calculated inertia load F_I is applied as horizontal load(s) at hull level in a quasi-static global analysis (ULS).

The Inertia load (F_I) as calculated below is considered to be an environmental load. It shall be scaled with the corresponding load factor for the LRFD method.

(i) Inertia load based on base shear (ULS)

The simplified procedure as outlined below will normally suffice /13/ for a quasi-static global analysis.

Because the instantaneous wave/current force resultant for a jack-up platform is not at the effective centre of mass, the dynamic amplification factor (DAF) in 4.4.4 may be regarded as a dynamic amplification factor for the total base shear, only. The base shear will typically be calculated by the Stoke 5th order theory.

Based on this amplified total base shear, "inertia" forces are derived which are to be applied at the platform effective centre of mass.

Inertia force $F_I = Q_A (DAF - 1)$

The simplified DAF is taken from case (i) in 4.4.4.

The procedure is illustrated in Figure 4-6.



 $Q_M =$ mean value of total base shear. $Q_A =$ amplitude value of total base shear (quasi-static analysis) $Q_A (DAF - 1) =$ inertia force (F_I) to be applied at the platform effective centre of mass in order to derive at maximum total responses.

Figure 4-6 Dynamic effects corrected by base shear

(ii) Inertia load based on hull displacement, Deterministic. (ULS)

This is a similar method as the base shear methods in (i). But it is based on hull horizontal displacement (Δ_A) from the wave/current loads in the considered wave direction instead of the wave load base shear. The hull displacement amplitude will typically be calculated by the Stoke 5th order theory. The calculated inertia load F_I is applied in a quasi-static global analysis.

It involves calculating the hull displacements caused by the fabrication, wind, wave and current loads to establish the hull displacement amplitude. The hull displacement may typically be chosen in the hull centre, and normally the displacements from the wave phase angles corresponding to maximum and minimum base shear can be used to determine the displacement amplitude.

In a global analysis the above may be obtained by applying a unit horizontal load (inertial load) at the platform effective centre of mass. The inertia load is obtained by scaling the unit load F_{Unit} with a scaling factor SFAC. See Figure 4-7.



Figure 4-7 Dynamic effects corrected by hull displacements.

Δ_{M}	= mean value of hull displacement.
$\Delta_{\rm A}$	= amplitude value of total hull displacement wave/current (quasi-static).
Δ_{Unit}	= the hull displacement from the unit force applied at the platform effective centre of mass
DAF	= The simplified DAF from case (i) in $4.4.4$.

SFAC = $\Delta_A (DAF - 1)/\Delta_{Unit}$

 Δ_A (DAF - 1) = the hull displacement representing the dynamic amplification Inertia force:

 $F_I = F_{unit} \times SFAC = F_{unit} \times \Delta_A (DAF - 1) / \Delta_{Unit}$

(iii) Inertia load based on hull displacement, Stochastic. (ULS)

The inertia load F_I may also be based on the stochastic dynamic amplification factor (DAF_S) based on case (ii) in 4.4.4.

The base shear or hull displacement amplitude will typically be calculated by the Stoke 5th order theory similar as in Case (i) and (ii) above, respectively. The stochastic dynamic amplification factor (DAF_S) may be used on both cases, i.e. select between:

Inertia force (i)
$$F_I = Q_A (DAF_S - 1)$$

Inertia force (ii)
$$F_I = F_{unit} \times \Delta_A (DAF_S - 1) / \Delta_{Unit}$$

The inertia force (i) above is assumed to be conservative but simpler to apply a global analysis.

4.4.7 Hull sway and P-⊿ loads (D and E-loads)

The P- Δ effect described in this section is based on that global analysis for the elevated condition is based on linear elastic finite element programs.

The jack-up is a relative flexible structure subject to hull sway (lateral displacements, e) mainly caused by environmental loads (E-loads).

Tolerances during fabrication will also lead to a "hull sway" (e_0), which is considered as a deformation load (D-load). Both e_0 and e are accounted for in the total hull sway Δ . The non-linear amplification factor α is also applied for the total hull sway. See further details below

Because of the hull sway Δ , the vertical spudcan reaction will no longer pass through the centroid of the leg at the level of the hull. The leg moments at the level of the hull will increase compared to those calculated by a linear quasi-static analysis. The increased moment will be the individual leg load P times the hull sway Δ .

(i) Horizontal offset (hull sway) hull from Fabrication and Installation (D)

Due to tolerances and heel of platform during fabrication, the hull may be horizontal shifted a distance e_0 relative to the legs at sea bottom. See Figure 4-8.

The value of e_0 should be specified by the designer. However, a value less than 0,005 l should normally not be accepted for the strength analysis.



 $e_1 =$ out-of-straightness. $e_2 =$ hull leg clearances.

 e_3 = heel of platform.

Figure 4-8 Horizontal offset of platform.

(ii) Hull sway from Environmental loads (E)

The overall hull deflections due to second order bending effects of the legs shall be accounted for whenever significant. The hull sway e from environmental loads is:

e = Sideway 1st order deflection of barge due to wind, waves and current include dynamic effects.

(iii) Hull sway (Δ) including non linear amplification factor (α)

The non linear amplification factor (α):

$$\alpha = \frac{1}{\left(1 - \frac{P}{P_E}\right)}$$

where

 P_E = Euler load as defined in Appendix A2. P = average axial leg load on one leg due to

 P^{-} = average axial leg load on one leg due to functional loads only (loads at hull level is normally sufficient). The total hull sway (Δ) including non linear amplification factor (α) may be taken as:

$$\Delta = \alpha (\gamma_{\rm f,D} e_0 + \gamma_{\rm f,E} e)$$

where

e_o = maximum horizontal offset of the platform, including initial out-of-straightness of legs

- e = Sideway 1st order deflection of barge due to wind, waves and current include dynamic effects.
- $\gamma_{f,D}$ = Load factor deformation loads
- $\gamma_{f,E}^{I,D}$ = Load factor environmental loads

(iv) $P-\Delta$ effect

In lieu of more accurate analysis, the P- Δ effect may be represented by a horizontal load (H) at hull elevation:

 $H = W\Delta / 1$

where

W = Elevated hull weight (3P)

P = Average load on legs at hull level

 Δ = total hull sway in the considered wave direction

1 = vertical distance between leg footing support point and the leg-to-hull connection.

The leg-to-hull connection can normally be set to the midpoint between lower guide and fixation elevations. See Figure 4-9.

The P- Δ effect may alternatively be included by the horizontal force H calibrated to give the moment M = P × Δ at the top of the legs. H can for a lattice leg be applied such that static equilibrium of the axial forces in the chords just below the lower guide equals the moment M = P × Δ . This method is expected to be most accurate of the two, it will require some more calculations to determine H. The equilibrium of axial forces may also introduce an axial force at top of leg in additions to M. In such case it may be necessary to correct the results for the axial leg force caused by H.

Please note that the horizontal load H in both cases above will give a false shear force in the legs. The shear force in the legs introduced by H need not be included in the check of leg strength. For example for a lattice leg, the force H should be included in check of the chords but it may normally be excluded when checking the braces and span breakers.



Figure 4-9 P-⊿ effect by horizontal loads

4.5 Global analysis for the transit condition

4.5.1 General

In the transit condition the legs are fully elevated and supported as cantilevers in the hull. Any rolling or pitching motion in combination with wind induces large bending moments in the legs and large reaction forces in jack houses and supporting hull structure. Consequently the critical areas in the transit condition are:

- Lower part of legs
- Jack houses
- Jacking machinery capacity
- Leg guides and porting hull structures
- Holding capacity of fixation system.

In addition to the leg forces, the hull is exposed to sea pressure.

In the transit condition the designer should clearly state if the legs are to be skimmed at the guides. If the legs are not to be skimmed then impact loading resulting from leg/guide shock loading is to be fully considered.

In general the legs are to be designed for static forces and inertia forces resulting from the motions in the most severe environmental transit conditions, combined with wind forces resulting from the maximum wind velocity. Wave motions may be obtained either from model tests or from computations. It should be emphasized, however, that a general wave motions computer program should be applied with great care. The proportions of a jack-up hull are so unusual that it will normally be necessary to apply a program which may account for three-dimensional effects. In addition the wave motion response will be influenced by non-linear effects such as:

- Non-linear damping
- Water on deck
- Bottom out of water.

Because the most severe motions are dominated by rolling or pitching at the natural period, the damping is essential for calculation of the maximum roll or pitch amplitudes. Non-linearities in the restoring forces and moments may on the other hand be responsible for a shift in the natural period.

4.5.2 Wave motion analysis for transit condition

If a rigorous wave motion analysis is carried out, see 4.5.1, the natural period of the roll or pitch motion may be determined as the period corresponding to the resonance peak of the transfer function /18/.

See in Appendix A10 for simplified wave motion analysis.

4.5.3 Global analysis for legs in transit

In lieu of more accurate analysis it is possible to resort to a simplified analysis procedure described in DNV-OS-C104. According to this procedure it is sufficient to consider the following loads:

- Inertia forces corresponding to a specified amplitude of roll or pitch motion at the natural period of the platform.
- Static forces corresponding to the maximum inclination of the legs due to rolling or pitching.
- Wind forces corresponding to a specified wind velocity.

The effect of heave, surge and sway are implicitly accounted for by use of a specified load scaling factor = 1.2. The application of this scaling factor is described in DNV-OS-C104 Sec.3B200.

For calculation of leg forces it is assumed that the roll or pitch motion can be described by:

$$\theta = \theta_0 \sin \frac{2 \pi t}{T_0}$$

where

t = time variable.

 T_o = natural period of roll or pitch.

 θ_0 = amplitude of roll or pitch.

The axis of rotation is assumed to be located in the water plane, see Figure 4-10.

The acceleration of a concentrated mass located at a distance r from the axis of rotation is then:

$$a = -\varepsilon_0 r \sin \frac{2\pi t}{T_0}$$

where

$$\varepsilon_{\rm o} = \left(\frac{2\pi}{T_{\rm o}}\right)^2 \theta_{\rm o}$$

The loads for the legs for a point with mass m in the leg structure will then be as shown in Table 4-1.

Table 4-1 Forces on a pointmass m in the legs during transit			
Force	Lateral forces (x- or y-direction)	Axial forces (z-direction)	
Static	$F_{TS} = mg \sin \theta_0$	$F_{LS} = mg \cos \theta_0$	
Inertia	$F_{TD} = m \epsilon_0 z$	$F_{LD} = m \varepsilon_0 y^{(1)}$ $F_{LD} = m \varepsilon_0 x^{(2)}$	
Wind	$F_{\rm W} = \frac{1}{2} \rho C_{\rm D} D [v(z) \cos \theta_0]^2$	N.A.	

¹⁾ for roll (rotation about x-axis)

²⁾ for pitch (rotation about y-axis)

The total loads are found by adding up the loads for each member calculated according to Table 4-1. This calculations may be time consuming if done by hand for a detailed leg model, but in some finite element programs the input is simplified and the detailed load calculations and load applications for each member are taken care of by the program.

It is sufficient with density and cross section area for the leg members to calculate the static loads.

The maximum rotation angle (θ_0) and roll or pitch period (T₀) is needed in addition for the inertia loads.

The wind load is similar to a current load calculation. The current velocity profile are replaced with wind velocity profile, the velocity profile is applied according to the transit arrangement and the air density is used for the drag forces. These type of calculations are also included in load programs used for wave and current load calculations.

The boundary condition as shown in Figure 4-12 may be used for checking leg strength and forces in the fixation system if the loads from Table 4-1 are applied to the legs.

Leg strength checked by simplified models is described in Appendix A9.





4.5.4 Hull barge in transit

The governing loads for the barge away from the leg support areas are normally hydrostatic pressures from sea for deck, bottom and sides, and tank pressures for bulkheads. Design formulas are specified in the DNV-OS-C101Sec4. In addition local loads from deck loads, drill floor, drill floor cantilever and machinery must be considered.

The governing loads for bulkheads and sides with deck and bottom flanges in way of the leg supports are normally a combination of maximum leg bending moments due to the barge motions and the hydrostatic loads. The overall bending of the barge due to waves are normally insignificant compared to the leg bending moments. To analyse the distribution of the leg bending moments into the barge, it is normally necessary to use a grillage beam model or a Finite Element Model to get accurate results. Conservative simplified model may, however, be applied.

The static response is caused by the permanent loads (lightship weight), variable functional loads and deformation loads. Variable functional loads on deck areas for global design are given in DNV-OS-C101 Sec.3. Global design load conditions should be established based on representative variable functional load combinations. For the jack-up in transit the response is experienced as a bending moment due to the buoyancy in the hull barge. Effect of different ballast distribution shall be adequately accounted for as part of the global load effects for the structural design of the hull structure. Maximum and minimum ballast conditions, representing unfavourable conditions used in the tank load plans, for sagging and hogging moments are typically shown in Figure 4-11. Effects with non-symmetric, diagonally distributed tank loading conditions should also be considered.



Figure 4-11 Ballast distributions in hull structure

4.5.5 Boundary conditions for a global jack-up model in transit

To avoid rigid body motion of a global structural model for the transit condition at least 6 degrees of freedom have to be fixed.

Fixed boundary conditions may be used for a statically determined set of boundary conditions while spring stiffness is more appropriate for a statically undetermined set of boundary conditions.

A statically determined set of boundary conditions is illustrated in Figure 4-12, with the following restraints:

- 3 vertical restraints (Z)

- 1 transversal horizontal restraints (Y)

- 2 longitudinal horizontal restraint (X).

Note that in the figure the two points with fixation in X must have the same X-coordinate and all three points must have the same Z-coordinate.

Any unbalance in loads of significance (hydrodynamic pressure vs. model acceleration) should be dealt with to avoid unphysical support reactions, i.e. the initial model should be in reasonable balance.

A statically undetermined set of boundary conditions may be represented with spring stiffness. The total vertical stiffness should be according to the water plane area. Using spring stiffness is appropriate when a significant unbalance in loads cannot be resolved. There should be enough springs so as to distribute the unbalance over several points. The higher unbalance the more springs. And these springs should be at 'strong' points in the model so as to limit the effect of unphysical support reactions.



Figure 4-12 Statically determined boundary conditions for Transit

4.6 Global analysis for the installation and retrieval conditions

In the installation and retrieval conditions, a leg may be subjected to an impact force from the sea bottom, due to the jack-up's motions in waves, see Figure 4-13.

Limiting environmental conditions (e.g. waves, current, wind, motions etc.) should be specified by the designer.

Normally the impact force may be assumed to be governed by rolling and pitching except for platforms with roll and pitch damping devices. The impact force due to a roll or pitch motion may be calculated by use of a simplified method based on the following conservative assumptions:

- Only one leg touches the bottom.
- The lower end of the leg is stopped immediately when the leg touches the bottom.
- The bottom is infinitely rigid.



Figure 4-13 Bottom impact on legs.

The rotational energy of the jack-up must then be absorbed by the leg and the porting structure at the barge. The impact force may be given as:

$$P_{H} = \frac{2\pi}{T} \theta \sqrt{\frac{I_{m}k_{H}}{\left[1 + \frac{k_{V}}{k_{H}} \left(\frac{d}{h}\right)^{2}\right]}}$$

$$P_{V} = \frac{2\pi}{T} \theta \sqrt{\frac{I_{m}k_{V}}{\left[1 + \frac{k_{H}}{k_{V}} \left(\frac{h}{d}\right)^{2}\right]}}$$

where

- = overall lateral stiffness of the leg, see Appendix A \mathbf{k}_{H}
- $\mathbf{k}_{\mathbf{V}}$ = overall vertical stiffness of the leg incl. vertical fixation at the barge.
- I_m T = mass moment of inertia of the jack-up with respect to roll or pitch motion.
- = period of roll or pitch, see 4.5.
- = amplitude of roll or pitch. θ

The mass moment of inertia may be evaluated in a way similar to that shown in 4.5.2. However, the actual position of the legs, and the added mass of the legs has to be considered. The result will depend on wave conditions and water depth.

The maximum allowable value of the impact force, P_{all}, may be determined from the appropriate strength criteria for the leg and porting structure at the barge.
5. Local structural analyses and considerations

5.1 Spudcans and lower part of leg - Elevated condition

(i) Spudcan and lower leg model for ULS and ALS

Yield and buckling checks of the spudcan and the lowest bay of the legs may be based on a local analyses model as shown in Figure 5-1. The suggested FEM model is most suitable for ULS and ALS checks.

Shell elements can be used for bulkheads, plating (top and bottom) and main girders of the spudcan. Stiffeners and secondary girders of the spudcan may be modelled with beam elements.

The leg members (chords, braces and span breakers) are modelled with beams as shown.

The connection between leg (beam elements) and spudcan (shell elements) is typically represented by a relative stiff "beam cross". The purpose is to transfer forces/moments from the beam chord centre point out to the "circumference" of the chord in the shell element model. The purpose is to secure that a plane normal to the chord remain plane after bending at the beam-to-shell transition.

The model should extend at least one bay above where the results are used for design checks, and chord nodes at top of the models are fixed.

The soil reactions (forces and moments) can typically be applied as pressure loads on the bottom plate of the spudcan, it is normally necessary to establish a number of loadcases to cover different combinations of vertical force and moment.

For separate type spudcans the maximum design eccentricity moment and corresponding critical design contact pressure may be based on DNV-OS-C104Sec.5 B200, /2/.



Figure 5-1 Typical model for spudcan and lower bay of leg

(ii) Detailed spudcan model included in global model for FLS

For fatigue model a refined mesh compared to ULS-model may be necessary in the most fatigue sensitive areas, i.e. in the leg chord and the leg brace at their connections to the spudcan. The element size in the fatigue locations to be checked is typically $t \times t$, where t is the plate thickness. See DNV-RP-C203, /5/, for further instruction on how to model around "hot spots" to analyse stresses to be used in fatigue calculations.

For fatigue (FLS), the spudcan model is considered to be more suitable if it is included directly as a part of the global analysis model. See Figure 5-2.

In such cases the boundary conditions may be applied as soil springs on the spudcan bottom plate. Representative soil stiffness, for vertical soil reaction and moment restraints is typically represented by vertical springs. These local springs can be modelled at bulkhead intersections of bottom plate, i.e. at relative stiff joints to avoid locally high un-realistic stresses in bottom plate.

The wave loads for fatigue is applied on the leg and normally there will not be necessary to apply loads on the spudcan in a fatigue analysis. The wave loads on a spudcan are very small and can be neglected. Thus it is not necessary to apply the hydro pressures on the spudcans.



Figure 5-2 Spudcan shell model as a part of the global model

(iii) Detailed spudcan model for stress concentration factors for FLS

As an alternative to include the local spudcan model directly as a part of the global analysis model, the Stress Concentration Factors (SCFs) may be determined from a local spudcan model as shown typical for a chord-to-spudcan connection in Figure 5-3.

Unit loadcases for axial force (N) and bending moments (M_{ip} and M_{oop}) can be applied to the model as shown in Figure 5-3. The loads may either be applied as concentrated force/moments as shown in Figure 5-3 an transferred by a stiff cross of beams to the shell model or applied as vertical line loads at top of the model.

The model extension from the load application to the fatigue locations selected for SCF calculations to be sufficiently to avoid local stress effect caused by the load application.

The boundary conditions may be applied as soil springs on the spudcan bottom plate as shown in Figure 5-2.

For each fatigue location to be checked the SCF and for each load case (N, M_{ip} or M_{oop}) may be taken as:

$$\text{SCF} = \sigma_{\text{principal}} / \sigma_{\text{nominal}}$$

where

N = Axial Force $M_{ip} = Bending moment in-plane$ $M_{oop} = Bending moment out-of-plane$ $\sigma_{principal} = maximum principal stress at the considered fatigue location for N, M_{ip} or M_{oop}$ $\sigma_{nominal} = nominal stress in chord (beam theory) N/A_{ch}, M_{ip}/W_{ipch} and M_{oop}/W_{oopch} for N, M_{ip} and M_{oop}$ $A_{ch} = cross section area of the chord$ $W_{ipch} = section modulus for in plane bending of the chord$

 W_{oopch}^{T} = section modulus for out-of plane bending of the chord

The SCF calculated above can finally be applied to scale the stresses in a global beam model for fatigue calculations at the fatigue locations to be considered.

In this global model the spudcan part can be simplified represented by coarse beam or shell elements. The spudcan part of the global beam model should be calibrated to have the same stiffness as the part subject to detailed analyses. The calibrated stiffness of the simplified part may be adjusted to obtain the same displacements/rotations for N, M_{ip} and M_{oop} .



Figure 5-3 Part of Leg chord to Spudcan model for SCF calculations

(iv) Sub-modelling for detailed spudcan model

Some FEM-programs also have possibility extracting displacements and rotations from a performed global analysis and apply these as prescribed displacements on the boundary of a detailed finite element model of the structure subject to detail stress analysis.

This method may be used for case (iii) above, i.e. displacements and rotations are applied at the positions for axial (N) for and bending moments (Mip and Moop).

Sub-modelling can be used be used for ULS, FLS and ALS conditions.

5.2 Leg structure

The distribution of shear forces and bending moments over the length of the legs is obtained from the global response analysis.

Forces in chords, braces and span breakers of lattice legs are normally found directly from a global analysis including a detailed leg model. For an equivalent leg model, see Appendix A.

In addition it is necessary to calculate the local distribution of stresses over the cross section, and in particular in areas where large forces are introduced.

(i) Tubular or box shaped legs

The stress distribution in tubular or box shaped legs, see Figure 4-1, may be determined from simple beam theory, combined with stress concentration factors when appropriate.

For local areas where large forces, such as reactions from guides or jacking mechanisms are introduced, it may be necessary to resort to finite element analysis.

(ii) Lattice legs

The local distribution of forces and bending moments in lattice legs, see Figure 4-1, is usually determined from computer analysis of a space frame model. Eccentricity of members in joints has to be accounted for. These effects may be included in the global model. Peak stresses (hot spot stresses) in the joints are only needed for the fatigue analysis, see 7.

(iii) Equivalent legs

Forces in chord and brace members from an equivalent leg analysis can be calculated as shown in Appendix A.

(iv) Guide reactions lattice legs

The introduction of guide reactions in lattice legs has to be considered in detail. Two aspects should be considered:

- Bending of the chord member due to guide reactions, see Figure 5-4 for illustration.
- Local deformation of the chord wall which is in contact with the guide, see Figure 5-4.

The bending action of the chord member may be obtained by leg beam model representing individual chord and brace members or by equivalent beam element analysis of the legs. The guide forces are assumed to be distributed over the guide height in a realistic manner.

The analysis of the chord wall may require detailed computer analysis considering the interaction between guide and chord wall.



Figure 5-4

Chord member subjected to guide reactions.

(v) Jacking reactions lattice legs

The introduction of jacking mechanism reactions has to be considered in detail. In cases where only a one-sided rack is used, see Figure 5-5, the horizontal component of the reaction has to be transmitted through the chord and bracing structure into the rack on the opposite chord. Since the horizontal reaction component may be very large, high bending stresses in the chords and high compressive stresses in the bracing will occur.

In cases where opposed racks are used, see Figure 5-5, the horizontal components are readily taken by compression of the individual racks, involving no bending of the chords or compression of the bracing.



Figure 5-5 Jacking mechanism reactions.

(vi) Leg Joint eccentricity

Members at a joint may be so designed that their axes do not intersect at one point. This represents an eccentricity which causes a bending action due to the eccentricity moment M_e , see Figure 5-6.

This may be accounted for in a detailed beam model of a lattice leg, where individual chords, braces and span breakers are represented by beams and the eccentricities at joints are modelled. If eccentricity is not modelled in such analyses, the moments may be added according to Figure 5-6.

For equivalent leg analysis the eccentricity can be included as shown in Appendix A



Figure 5-6 Joint eccentricity moments.

(vii) Span breakers Fatigue considerations (FLS)

For fatigue strength of spanbreakers the following local effects are sometimes not included in a global leg model:

- Spanbreakers in splash zone will be subject to variable buoyancy loads and wave forces during one wave cycle
- Vortex shedding from wind on spanbreakers in air and from current in submerged spanbreakers.

For a stochastic linear fatigue analysis as described in 7.7 the wave loads are based on first order wave theory, and the results are normalized with respect to wave height before being used in stress calculations, i.e. linearized frequency domain analysis.

The wave forces are typically calculated by integration to the top of the linearization wave heights, i.e. wave forces are calculated for the full wave period rather than considering that the members in the splash zone may be out of water for a part of the wave period. This part of the loading assumption is on the conservative side.

In such cases variations of span breaker wave force and buoyancy force when members come out of water will not be included a global leg model. This part of the loading assumption is on the non-conservative side.

The effect of the wave and buoyancy loading on span breakers in the splash zone depends on the position of the span breaker relative to the water level.

Fatigue strength of spanbreakers in air can also be subject to vortex shedding due to wind loads. Vortex shedding of submerged spanbreakers due to current velocities is less probable.

5.3 Leg joints by FEM analyses

Local shell/solid model of a leg joint may be included in the global beam analysis as shown in Figure 5-7. One of the detailed joint models is shown in Figure 5-8. Linear elastic analysis for this purpose can be suitable for FLS for cases where parametric SCFs are not available.

For ULS/ALS a non-linear analysis will normally be more suitable to be able to account for plastic strain.



Figure 5-7 Detailed FEM-model of joints in global analyses



Figure 5-8 Detail of FEM-model of joint

5.4 Hull and jack house

A local shell model of the hull structure will typically include the following parts:

- Bottom plate
- Main deck
- Intermediate decks (for example: double bottom plate)
- Primary girders/bulkheads
- Bulkheads in Leg well area, i.e. near hull connection to the legs.

Secondary girders and stiffeners may typically be modelled as beam elements, lumping of more than one stiffener or secondary girder to a mesh line is normally allowed.

The mesh size for ULS and ALS checks may typically be the same size as the girder (or frame) spacing.

For FLS the mesh needs to be considerably refined in areas where the stresses will be used in fatigue calculations. Typically element size $t \times t$ may be required. See DNV-RP-C203 for further guidelines.

The boundary conditions for a local model of the hull may best be accounted for by including the hull in the global model. Example of ULS/ALS model is shown in Figure 5-9, with enlarged jack house shown in Figure 5-10.

The hull structure around the leg-to-hull connections should be prepared for more than one elevation to represent the load distribution at guides, fixation systems and jacking pinions. See example of model for this connection in Figure 5-11.

Each of the connections shown in Figure 5-11 may typically be modelled as a cross of beams. The node in the centre of the cross may be connected to the leg (leg beam model) while the "outer nodes" of the cross can be connected to the hull/jack house (shell model).

The transfer of relevant forces and force directions between hull and leg is different for guides, pinions and fixation systems. Through the guides typically horizontal force(s) in one or two directions are typically transferred. The fixations system normally transfers horizontal and vertical forces, while pinions typically transfer vertical forces or noting (de-coupled). This can in the model can be controlled by introducing hinges in relevant directions or about relevant axes. Hinges is a way to reduce which translations and/or rotations at selected beam ends of the cross that will be connected and thus will transfer forces. Adjusting selected stiffness properties may also be used to obtain load transfer in relevant directions.

The stiffness of the cross beams can also be adjusted if the flexibility at a connection is to be represented in the model. For example if the jacking pinions are "softly" supported by rubber pads, the stiffness of the cross beams can be adjusted to account for this.

The leg chords of the leg model are typical beam elements that need to be split into shorter beams than one bay height due to the connection points.



Figure 5-9 Hull and jack house shell model as part of the global analysis



Figure 5-10 Enlarged jack-house as part of the hull shell model



Figure 5-11 Example of leg-to-hull connections for hull shell model

6. Structural strength (ULS)

6.1 Introduction

Structural utilisation shall be evaluated in accordance with the requirements of the limit states ULS, FLS, and ALS as referred to in DNV-OS-C104 and DNV-OS-C101.

Guidance on the practical application of the general strength criteria may be found in Recommended Practices, Classification Notes, design codes or other recognized publications. The characteristic static strength for ULS may be calculated according to recommended codes listed in DNV-OS-C101Sec.5 for:

- Flat plated structures and stiffened panels
- Shell structures
- Tubular members
- Tubular joints
- Conical transitions
- Non-tubular beams, columns and frames.

The following modes of failure to be considered:

- Excessive yielding (ULS, ALS).
- Buckling (ULS, ALS
- Brittle fracture.
- Fatigue fracture (FLS). See more details in 7.

The above limit states refer to the load and resistance factor design format (LRFD format) with use of material-(resistance) and load-factors as safety factors.

The typical check in LRFD is that Usage Factor (UF) is lower than allowable usage factor (UF_{allowed} = 1.0), i.e.

$$UF = S_d / R_d \le 1.0$$

where

 S_d = Design load effect

 R_d = Design resistance

The above strength checks are based on the LRFD-format. For working stress design format (WSD) the loads are without load factors and the utilisation is checked against permissible usage factors (η). See sections 1.5 and 1.6 for LRFD and WSD design formats, respectively.

Stresses and stress resultants obtained from the global and local response analysis have to be checked against relevant strength criteria.

In the static strength analysis only the quasi-static effect of variable loads is considered. The effect of load variation is considered in the fatigue strength analysis, see 7.

(i) Excessive yielding

Structural members for which excessive yielding is a possible mode of failure, are to be investigated for yielding. Local peak stresses from linear elastic analysis in areas with pronounced geometrical changes, may exceed the yield stress provided that the adjacent structural parts has capacity for the redistributed stresses.

Local yielding due to a stress concentration may be accepted, see Figure 6-1. The effect of stress concentrations is only considered in the fatigue analysis.

Full advantage is not always taken of the real load carrying capacity, see Figure 6-2. However, it is allowable according to the Rules to use ultimate strength analysis in special cases.



Max. stress: $\sigma_{M} = SCF \times \sigma$ First yield: $\sigma = \sigma_{F} / SCF$ Plastic failure: $\sigma = \sigma_{F} (1 - D/B)$

Nominal design stress: $\sigma_{\rm N} = \frac{\sigma}{1 - \frac{D}{R}}$

Figure 6-1 Stress concentration due to a cut out.



Max. stress: $\sigma = \frac{PL}{2\pi D^2 t}$

First yield: P = $2 \pi \sigma_F D^2 \frac{t}{L}$

Plastic failure: P = 8 $\sigma_{\rm F} D^2 \frac{t}{L}$

Figure 6-2 Ultimate strength of a beam.

(ii) Buckling

The possibility for buckling has to be considered for all slender structural members. When buckling is a governing mode of failure, it is essential that geometric imperfections are kept within specified limits.

(iii) Brittle fracture

The possibility for brittle fracture shall be avoided by the structural categorisation, selection of materials, and suitable inspection, as specified in DNV-OS-C104 Sec.2 and DNV-OS-C101 Sec.4. Brittle fracture is normally not treated as a design criterion. Guidance to avoid brittle fracture is given in DNV-OS-C101 Sec.4 C100.

6.2 Local strength of legs

The local strength of legs may be evaluated using the general strength criteria referred to in 6.1. The governing criterion will always depend on the actual design.

(i) Tubular and box shaped legs

Tubular and box shaped legs, see Figure 4-1, are designed as stiffened plate or shell structures. The critical stress, σ_{cr} , is the value of the axial stress at which local failure will occur. Such legs are normally proportioned and stiffened in such a way that local buckling of plates and stiffeners is excluded. The critical stress will then be determined by the yield criterion as:

$$R_{d} = \boldsymbol{\sigma}_{cr} / \boldsymbol{\gamma}_{m} = \frac{\left(\frac{\boldsymbol{\sigma}_{x}}{\boldsymbol{\sigma}_{e}} \boldsymbol{\sigma}_{F}\right)}{\boldsymbol{\gamma}_{m}}$$

where: σ_x and σ_e are the actual values of the axial stress component and the von Mises equivalent stress, respectively. However, it will always be necessary to verify that local buckling is not possible.

(ii) Lattice legs

The local strength of lattice legs depends on:

- Strength of individual members.
- Strength of joints.
- Local strength of members subjected to reactions from guides and jacking mechanisms.

(iii) Chord members

The critical axial stress of a chord member between two bay nodes can be found by considering the chord as a column or a beam-column. The effective length factor for chord members should not be taken less than: K = 1.0 The local strength of the chord wall has to be specially considered in each case, depending on the actual design

of guides, wedges, clearances etc., see Figure 5-4. For jack-ups with height of guides less than distance between chord joints, local bending should be accounted for in check of leg chord strength. The leg chords are to be considered for two guide and/or chock positions as shown in Figure 5-4.

(iv) Brace members

The strength of bracing members may be evaluated using the general buckling criteria for beam-columns. The effective length factor for bracing members should normally not be taken less than: K = 0.8

However, a reduced value may be obtained by use of the diagram shown in Figure 6-3. Having determined G_A and G_B for end A and end B of the bracing member under consideration, K is obtained by constructing a straight line between the appropriate points on the scales for G_A and G_B The general definition of G_A and G_B is given in Classification Note No. 30.1 /9/ and in /21/. However, for in-plane buckling of horizontal and diagonal bracing members in a jack-up leg, see Figure 6-4, the following values may be used:

$$G_{A} = 1.0$$

$$G_B = 15 \left(\frac{r}{R}\right)^3 \left(\frac{t}{l}\right) \left[\frac{T}{R} - 0.01\right]^{(1.5r/R - 2.35)}$$

where

- 1 =length of bracing member.
- r = radius of bracing member.
- t = wall thickness of bracing member.
- R = radius of chord member.
- T = wall thickness of chord member.



Example:

If $G_A = 1,0$ and $G_B = 0,3$ then K = 0,7

Figure 6-3

Evaluation of the effective length factor, K.



Figure 6-4 Buckling of bracing members.

6.3 Tubular joints

Joints with parameters outside the validity range given for the various formulas are to be subject to detailed investigations. The validity range for the parametric formulas depends on the following ratios and parameters:

β , γ , θ and D/g (for K-joints) as defined below.

The following values are typically defined for tubular joints:

- T = wall thickness of chord.
- t = wall thickness of considered brace.
- R = outer radius of chord wall.
- r = outer radius of considered brace wall.
- θ = angle between chord and considered brace.
- D = outer diameter of chord.
- d = outer diameter of brace.
- a = gap between considered brace and nearest load-carrying brace, measured at the surface of the chord.
- $\beta = \tilde{r}/\tilde{R}$
- $\dot{\gamma} = R/T$
- $\dot{g} = a/D$

(i) Simple joints

It is recommended that simple tubular joints without overlapping are designed in accordance with generally accepted standards and codes which are based on the following principles:

- If an increased wall thickness, or special steel, in the chord at the joint is required, it should be extended
 past the outside edge of the bracing a minimum of one quarter of the chord diameter or 300 mm, whichever
 is greater. See Figure 6-5.
- Where increased wall thickness or special steel is used for braces in the tubular joint area, it shall extend a
 minimum of one brace diameter or 600 mm, whichever is greater, from the joint. See Figure 6-5.
- Nominally concentric joints may be designed with the working points (intersections of brace and chord centrelines) offset in either direction by as much as one quarter of the chord diameter in order to obtain a minimum clear distance of 50 mm between non-overlapping braces, or to reduce the required length of heavy wall in the chord. See Figure 6-5.

(ii) Static strength of multi-brace joints

Multi-brace joints without stiffening or overlaps may, in lieu of more accurate treatment, be assessed according to formulas for simple welded tubular joints. Care is to be taken to ensure that the formulae are used in a conservative way. The characteristic strength of individual braces is in no case to be taken larger than the values given by simple welded tubular joints.

When applying the simple joint formulae for multi-brace joints, the main concern is whether individual brace loads are additive by tending to deform the chord in the same way or opposing. Brace loads in different planes tending to give opposing deformations may be disregarded, whereas brace loads in different planes amplifying the chord deformations induced are to be superimposed.



Figure 6-5 Detail of simple joint.

(iii) Overlapping joints

Overlapping joints, in which part of the load is transferred directly from one brace to another through their common weld, should be designed as follows:

The overlap shall preferably be proportioned to transfer at least 50% of the allowable total load component
perpendicular to the chord.

- The brace wall thickness shall in no case exceed the chord wall thickness.
- Where the braces carry substantially different loads and/or one brace is thicker than the other, the heavier brace shall preferably be the through brace with its full circumference welded to the chord. See Figure 6-6.



Figure 6-6 Detail of overlapping joint.

In cases where stiffeners must be introduced to increase the chord strength, due attention should be paid to stiffener design and positioning to avoid serious local stress concentrations.

To accept these joints, comprehensive documentation revealing adequate static and fatigue strength will be needed.

Where bracing members in adjacent planes tend to overlap in congested joints, the following corrective measures may be considered:

- Where primary braces are substantially thicker than the secondary braces, they may be made the through members, with the secondary braces designed as overlapping members. See Figure 6-7a.
- The chord may be given an enlarged joint section as indicated in Figure 6-7b.
- A spherical joint may be used. See Figure 6-7c.
- Secondary braces causing interference may be spread out as indicated in Figure 6-7d.

(iv) Eccentric braces

Moments caused by eccentricity of the brace working lines are to be considered in the structural analysis when the eccentricity is greater than D/4.

(v) Static strength of complex joints

In lieu of detailed FEM-analysis, complex joints may be controlled for adequate static strength by cutting sections which isolate groups of members, individual members, and separate elements of the joint (e.g. gussets, diaphragms, stiffeners, welds in shear, surfaces subjected to punching shear), and verifying that a distribution of stress may be assumed that satisfactory equilibrium without exceeding the design strength of the material.





6.4 Hull (Deck) structure

The deck plates and/or bulkheads may be designed according to one of the following approaches to calculate plate stresses:

- Isotropic plate stiffness, i.e. the deck plates carries normal stresses (σ_x and σ_{y}) and shear stress (τ_{xy}). See (i) below.
- Anisotropic plate stiffness, i.e. the deck plates carries shear stress (τ_{xy}), but not normal stresses (σ_x and σ_y). See (ii) below.

The stresses carried by the selected approach are also to be accounted for in the design checks.

Stresses from local and global models are to be superimposed to account for that all local loads are not included in the global hull shell model. See (iii) below.

(i) Design based on isotropic plate stiffness

Figure 6-8 shows a typical part of deck structure, with the deck divided into the following structural categories:

- Primary girders/bulkheads
- Secondary girders
- Stiffeners
- Plates.



Figure 6-8 Typical part of deck structure

Primary girders/bulkheads

The primary girder and the main bulkheads with effective part of deck plating/heavy flanges take part of the global stiffness and strength of the deck structure. Such primary girders will be subject to both global and local responses, i.e. the stresses from the global analyses model should be superimposed with the girder bending stresses caused by the local deck loads and tank pressures.

The effects of cut-outs shall be considered. Large cut-outs/openings are normally included in the global analysis model.

Secondary girders

The secondary girders are supported by the primary girders/bulkheads.

In cases where the secondary girders are taking part in the global strength of the deck structure, these girders shall be designed for the combined effect of global and local loads, including girder bending stresses caused by the local deck loads. Buckling of the panel, comprising girders, stiffeners, and plate, shall be considered.

In cases where the secondary girders, plate, and stiffeners are not taking part in the global strength, analysis and design based on the anisotropic plate stiffness may be applied, see below for conditions and recommendations.

Plate and stiffeners

All the various stress components (local and global) should normally be evaluated and the relevant combination of stresses to be checked against the buckling and yield criteria.

(ii) Design based on anisotropic plate stiffness

Analysis with anisotropic plate stiffness (also referred to as "stressed skin" design philosophy) may be applied to the analyses and design of deck structures. The philosophy implies specific requirements for both the global model as well as the local model(s) as referred to below.

For the deck structure, the 'stressed skin' philosophy may be applied to large deck areas in-between primary girders/bulkheads. The stressed skin elements will represent plate panels that only resist shear forces in the global analysis model. This means that all membrane stresses, both tensile and compression stresses, are ignored in the panels. The purposes of introducing stressed skin elements is to let primary girders/bulkheads and trusses (including thick deck plates representing heavy flanges close to the web) have sufficient strength to take the global loads. The deck plates in-between will be designed to resist local loads and shear forces from global analysis. Hence the structural design is based on the following basic assumptions:

- a) Plate panels with stiffeners are only assumed to resist global shear stresses in plate and local loads.
- b) Secondary girders are assumed to resist local loads.
- c) Shear forces may be redistributed to obtain equal shear flow over the total panel length.
- d) Primary girders/bulkheads/trusses (including heavy flanges in decks) carry the normal stresses from global analysis model. These structures are treated with normal isotropic material properties in the global analysis, and will take care of the global strength integrity of the upper hull deck structure (ULS).
- e) Stressed skin elements may be modelled by adjusting the material matrix for global analysis (ULS). Adjustments may be performed by using anisotropic material model, for example: maintaining the shear stiffness and divide the axial stiffness by 100.
- f) Note that fatigue evaluation based on analysis with stressed skin elements will be non-conservative for the stressed skin elements (see item e above).

Hence global analysis for fatigue assessment shall be performed with isotropic material model (axial stiffness not modified).

(iii) Superimposing responses hull structure

The simultaneity of the responses resulting from the local and global analysis models, including various sea and tank pressures, may normally be accounted for by linear superposition of the responses for logical load combinations.

When evaluating responses by superimposing stresses resulting from different models, consideration shall be given to the following:

- Loads applied in global and local models
- It should be ensured that responses from design loads are not included more than once.

6.5 Spudcans

The main structure of spudcans is typical constructed as plated structures with girders and stiffeners:

- Bottom plate
- Top plate
- Vertical bulkheads in radial and/or circumferential directions
- Outer side wall
- Skirts (not for all spudcans).

The approach of design will be similar as for the hull structure described in 6.4.

The capacity of a spudcan at connections to leg chords and braces are to be checked for yield and buckling strength, as they transfer large forces/moments from the above leg structure.

Local water pressure for the spudcan is normally not governing the design checks but the load effect should be clarified:

- Normally it is not necessary to include local water pressure if the spudcans are free flooded, i.e. the water give equal but opposite directed pressures on inner and outer sides of the spudcan plating. In such cases the net pressure is zero and need not be accounted for.
- For conditions without free flooding, the local water pressure is to be accounted for.

The strength check can be based on stresses from a finite element analysis as described in 5.1. The local soil pressure on bottom plate is to be applied to represent different the eccentric vertical soil reactions, i.e. to cover different force/moment combinations.

6.6 Capacity for jacking and fixation systems

Forces transferred through the jacking and fixation system can normally be checked against capacities provided by the manufacturers of the systems. These components may be subject to separate certification by DNV.

Normally the capacities are specified as safe working loads (SWL), i.e. they include the necessary safety factors, and the forces (FORCE) calculated in WSD-format (without load factors) can normally be used directly to check the capacity. The capacity is OK if: FORCE \leq SWL.

For LRFD-format, the forces with load factors is conservative when compared to SWL, and the forces may be adjusted for load factor = 1.0 on all loads.

It is necessary to clarify the basis for the presented capacities and also the interaction between jacking and fixation system during different conditions before doing the capacity checks.

(i) Capacity check for Fixation system

The calculated forces going through the fixation system from the global analysis may be used to check if the capacity of the fixation system is satisfactory.

Transit, installation and elevated conditions should be considered.

(ii) Capacity check for Jacking system

For the jacking system with the pinions engaged the following scenarios should be considered:

- Lifting the hull. The hull weight is completely carried by the jacking pinions during the jacking-up condition (ULS).
- Supporting the hull alone, i.e. no separate fixation system in elevated condition. Brakes and/or chocks used to lock the pinions at their position after elevating to final position (ULS, ALS).
- Supporting the hull in combination with a separate fixation system. After elevated to final elevated condition the pinions will support the hull weight, while the load changes thereafter will mainly be carried by the fixation system. The load distribution will depend on the relative stiffness between fixation system and jacking system.

The capacity of the jacking system should be considered for transit, jacking installation and elevated conditions.

7. Fatigue Strength (FLS)

7.1 Introduction

Requirements to the Fatigue Limit States (FLS) are given in DNV-OS-C104 Sec.6 and DNV-OS-C104 Appendix A and DNV-OS-C101 Sec.6. Methods and guidance concerning fatigue calculations are given in DNV-RP-C203.

A fatigue strength analysis is required for members and joints for which fatigue fracture is a possible mode of failure. For a typical jack-up the following fatigue locations are normally to be checked:

- Spudcan and Leg at the leg-to-spudcan connection
- Leg structure at and around the splash zone
- Leg and Hull/Jacking structure at and around the leg-to-hull connections.

Currents may normally be excluded from the fatigue analysis (except when such currents may induce significant vortex shedding effects).

In fatigue analysis it is the stress range that is needed, and not the total stress.

It is assumed that the Miner cumulative damage law is valid.

Fatigue was earlier not considered as a very important problem for jack-up platforms. The main reasons for this are that most platforms have been designed for areas with small or moderate wave heights and that the members subjected to high dynamic loads are not the same from location to location. Compared to fixed platforms, jack-up platforms also have the advantage that members subjected to fatigue damage may be regularly inspected and repaired.

For different reasons fatigue may be found to be important for some present and future designs, and thus has to be considered at the design stage:

- Jack-up platforms are used in areas with more severe sea conditions. This means that the wave loads which are most important in connection with fatigue, become more dominating.
- Jack-up platforms are used in areas with deeper water. This may lead to more flexible structures which
 means that the dynamic amplification of stresses will increase.
- In order to meet the requirements for more severe sea states and deeper water, the strength of the legs must be increased. Due to different considerations in particular regarding the transit condition, it is essential that the leg weight is minimized. This leads to the use of high strength steels. The design allowable stresses then increase, and the possibility for fatigue failure increases.
- Jack-up platforms may be used as fixed platforms for longer periods. The special advantages of a mobile unit with respect to inspection may then be lost.
- Fatigue is also important for ageing jack-up units, i.e. units with nominal age equal to or higher than the documented fatigue life.

7.2 Stress concentration factors

The stresses which are significant in a fatigue analysis are the hot spot stresses. The hot spot stress range is obtained by use of an appropriate stress concentration factor on the nominal stress.

(i) Tubular Joints

Hot spot stresses for tubular joints are normally defined as the stresses in the position which are as close to the weld as possible without being influenced by the weld profile. Hot spot stresses may be obtained by multiplying the nominal stresses by appropriate stress concentration factors (SCF). The hot spot stresses in tubular joints in lattice legs are thus given by:

Hot spot stress on brace: $\sigma_{\text{brace}} = \sigma_{\text{max}} \times \text{SCF}_{\text{brace}}$ Hot spot stress on chord: $\sigma_{\text{chord}} = \sigma_{\text{max}} \times \text{SCF}_{\text{chord}}$

where

 $\sigma_{\rm max}$ = nominal stress in the brace.

 SCF_{brace} = stress concentration factor for the brace.

 SCF_{chord} = stress concentration factor for the chord.

Stress concentration factors may be obtained from experiments or finite element analysis. Based on systematic investigations empirical formulae have been developed, see DNV-RP-C203.

However, formulae given in different publications may be in considerable disagreement, and they should be used with great care, and not outside their specified range of validity

(ii) Plated structures

Stress concentration factors for plated structures are given in DNV-RP-C203, for example:

— Butt welds for leg chord members with thickness transitions, including eccentricity.

- Holes
- etc.
- (iii) FEM-analysis for SCF calculations

See 5 and DNV-OS-C203 for how to calculate SCFs by FEM analysis.

7.3 Design fatigue factors (DFF)

The applications of Design Fatigue Factors (DFF) are discussed in DNV-OS-C101 Sec.6. The calculated fatigue life shall be longer than the design fatigue life times the DFF. The value of DFF = 1-3 provided the member/joint is redundant.

DFF shall be applied as fatigue safety factors for permanently installed units, as specified in DNV-OS-C101 Sec.6 A200 and DNV-OS-C104 Appendix A.

7.4 S-N curves

The S-N curve defines the allowable number (N) of constant stress cycles for a stress range ($\Delta\sigma$). It is assumed that the S-N curve is represented by:

$$\log N = \log \overline{a} - m \log \left[\Delta \sigma \left(\frac{t}{t_{ref}} \right)^k \right]$$

where

- m = negative inverse slope of the S-N curve, typically two slopes: m_1 and m_2 . log \bar{a} = intercept of log N axis
- t_{ref} = reference thickness equal 25 mm for welded connections other than tubular joints. For tubular joints the reference thickness is 32 mm. For bolts $t_{ref} = 25 \text{ mm}$
- = thickness through which a crack will most likely grow. $t = t_{ref}$ is used for thickness less than t_{ref} .
- = thickness exponent on fatigue strength. k

SN-curves with slopes (m₁ and m₂), intercept (log \overline{a}) and thickness exponents (k) are defined in DNV-RP-C203. Use of one slope S-N curves leads to results on the safe side.

Representative SN-curves for different weld details are also presented in DNV-RP-C203.

S-N curves are given for:

- components in air
- components in seawater with cathodic protection
- components in seawater for free corrosion
- tubular joints
- cast nodes
- forged nodes
- stainless steel.

7.5 Allowable extreme stress range

The design fatigue life for structural components should be based on the specified service life of the structure, with service life minimum 20 years. If the 100 year return period is used as basis for the analyses, the extreme stress range for 20 year return period (i.e. 10⁸ cycles) may be taken as:

$$\Delta \sigma_{20} = 0.92 \, {}^{1/h} \Delta \sigma_{100}$$

 $\begin{array}{l} \Delta\sigma_{20} = \text{extreme stress range during 20 years (10^8 \text{ cycles})} \\ \Delta\sigma_{100} = \text{extreme stress range during 100 years (10^{8.7} \text{ cycles})} \\ \text{h} = \text{Weibull stress range shape distribution parameter; see 7.8.} \end{array}$

Design charts for steel components for allowable extreme stress ranges are given in DNV-RP-C203 item 2.14.2, for components in air and components in seawater with cathodic protection. These charts have been derived based on two slopes S-N curves, and assumption of design fatigue life of 20 years (10⁸ cycles).

Note that the allowable extreme stress ranges should be reduced for longer design fatigue lives, DFFs and thickness effects.

7.6 Different operating conditions

The stresses in a particular member subjected to fatigue damage may be sensitive to changes in the actual operating conditions, for example wave direction, water depth, penetration, airgap etc for the elevated condition. Transit conditions should also be considered.

If a number of different conditions can be defined, the accumulated damage may be calculated as:

$$D = \Sigma n_i D_{1i}$$

where

 Σ = indication of summation over all different conditions.

 D_{1i} = annual accumulated damage for condition (i). n_i = number of years of endurance of condition (i). It is required that: $\Sigma n_i = 20$.

7.7 Stochastic fatigue analysis

Rather than using discrete waves, the various sea states may be described by wave spectra, and all frequency components in the sea state are represented. The long term distribution of waves may be described by a set of wave spectra, with varying significant wave height, period and probability of occurrence. By combining these spectra with the transfer function for member end stresses, which may have been calculated considering dynamic behaviour of the platform, the probability distribution of stress fluctuations for each sea state is obtained. The total fatigue damage is thereafter arrived at by summing up the contribution from each sea state, taking into account their probability of occurrence.

The complete procedure is outlined below:

a) For each wave direction considered, determine wave load transfer function

In practice, this is done by computing wave loads for a number (usually 10-30) of discrete wave frequencies. A wave steepness is given as input for each frequency, so that the loads may be computed for a wave of finite height. First order wave theory is used, and the results are normalized with respect to wave height before being used in stress calculations, see also 4.4.

b) Compute member end stress transfer functions

A structural dynamic analysis (steady state analysis, frequency domain) is carried out for each wave frequency. Stresses are computed for a number of points around each member end specified, and the various components (axial, shear, bending) are combined with due consideration to phase differences.

c) Introduce stress concentration factors

As described in 7.2 the stresses required for the fatigue analysis are the nominal stresses multiplied by the appropriate stress concentration factors. The stress concentration factors may vary within wide limits, depending on geometry and type of loading.

d) Computation of long term distribution of stresses

A number of sea spectra are specified by information on significant wave height, zero-upcrossing period, parameters related to spectral type and shape, and probability of occurrence. This information may be given as shown in Fig. 3.3. The effect of short-crested ness of the sea may be taken into account by considering a number of elementary wave directions associated with every main direction. The response spectra are determined numerically by combining the sea spectra and the transfer functions.

e) Determine usage factors

Relevant S-N curve and parameters describing the individual stress range distribution for each response spectrum are specified. The stresses may be assumed to be Rayleigh or Rice distributed. The number of cycles at a given stress level in one particular sea state may thus be calculated, and consequently the contribution to the fatigue damage at that level. The contribution from each sea state is calculated by considering all stress levels in the individual distribution, and the accumulated fatigue damage is found by summing up the contribution from each sea state.

7.8 Simplified fatigue evaluation

For the practical application of the methods, the problem is to establish the long term distribution of the stresses which are significant for the fatigue life prediction.

The result of a fatigue analysis is usually presented in terms of accumulated damage or fatigue life. Based on some simplifying assumptions these quantities may be given by explicit expressions.

It is assumed that the long term distribution of stresses is represented by a two-parameter Weibull distribution:

$$Q(\Delta \sigma) = \exp\left[-\left(\frac{\Delta \sigma}{q}\right)^{h}\right]$$

The parameters involved in these equations are:

N = number of cycles to failure.

Q = probability for exceedance of the stress range $\Delta \sigma$

 $\Delta \sigma$ = stress range.

h,q = Weibull parameters.

The accumulated fatigue damage is then given by /17/:

$$D = \frac{N_o}{\overline{a}} q^m \Gamma \left(\frac{m}{h} + 1\right)$$

where Γ is the gamma function and N_o is the number of cycles in the period under consideration. For a period of 20 years, $N_o \sim 10^8$.

This equation is linear with respect to N₀, and the accumulated fatigue damage in one year is then:

$$D_1 = D / 20$$

The fatigue life in years is:

$$T = 1 / D_1 = 20 / D$$

The Weibull parameter, q, may be related to the stress range ($\Delta \sigma_0$) with return period equal to 20 years:

$$q = \frac{\Delta \sigma_o}{\left(\ln N_o\right)^{1/h}}$$

The following expressions are then obtained for accumulated fatigue damage (D) and fatigue life (T):

$$D = 18,42^{-m/h} 10^8 \frac{\Delta \sigma_o^m}{\overline{a}} \Gamma\left(\frac{m}{h} + 1\right)$$
$$T = \frac{2 \overline{a} 18,42^{m/h}}{10^7 \Delta \sigma_o^m} \Gamma\left(\frac{m}{h} + 1\right)$$

The stress range, $\Delta \sigma_0$, with return period equal to 20 years, which will be associated with an accumulated damage D is:

$$\Delta \sigma_o = 18,42^{1/h} \left[\frac{\overline{a} \text{ D}}{10^8 \Gamma\left(\frac{m}{h} + 1\right)} \right]^{1/m}$$

The stress range $\Delta \sigma_0$ includes the geometrical stress concentration factor as described in DNV-RP-C203.

The maximum allowable stress range with return period equal to 20 years is obtained by introducing the maximum allowable accumulated fatigue damage for the same period.

Considering the idealized expressions for fatigue damage and fatigue life given above, it is obvious that the accuracy of a fatigue analysis is extremely sensitive to errors in the estimation of the long term stress distribution, see also /22/. This is illustrated by an example shown in Figure 7-1. Investigations have shown that even larger scatter is associated with the derivation and selection of S-N curve for each particular case /23/.

On this background it is obvious that even the most sophisticated fatigue analyses will be associated with considerable uncertainties. If one or more of the significant parameters are given only by approximate values, a simplified fatigue analysis based on conservative assumptions may be the most reasonable approach.



Figure 7-1

Relation between fatigue damage and the Weibull parameters $\Delta \sigma_0$ and h.

A simplified fatigue evaluation is based on the idealized equations given above. For practical use the following parameters are needed:

The stress range ($\Delta \sigma_0$) with return period equal to 20 years, or the corresponding stress amplitude ($\sigma_{amp} = \Delta \sigma_0 / 2$)

The Weibull parameter (h) which is associated with the shape of the long term distribution. See 7.9.

The stress range ($\Delta \sigma_0$) refers to hot spot stresses, which means that the corresponding nominal stress range has to be multiplied by an appropriate stress concentration factor.

The nominal stress range may normally be determined from the global response analysis required for the static strength analysis (ULS). In addition it is necessary to distinguish between the static part and the dynamic part of the stresses. In this connection the interaction between wave and current loads must be considered.

Corrections for short crested sea, actual damping ratio and varying water depth may be given as:

— Short crested sea:

To account for short crested sea, the fatigue life N' or acceptable stress range $\Delta \sigma_0$ may be analysed as /26/: N' = N (4/3)^{m/2}

Or

$$\Delta \sigma_{\rm o}' = 1,15 \Delta \sigma_{\rm o}$$

— Actual damping:

The damping assumed in Figure 7-3, 8% may be considered as the stochastic damping, i.e. the actual damping is about $\xi = 4\%$. See 4.4.6.

For other values of damping ξ the fatigue life N'' or acceptable stress range $\Delta \sigma_0$ '' may be analyses as /26/:

$$N'' = N (\xi/\xi)^{m/2}$$

or
$$\Delta \sigma_0'' = \Delta \sigma_0 (\xi/\xi)^{1/2}$$

— Varying water depth:

Because a jack-up normally will be operating at different water depths, the fatigue analysis should reflect this fact. A simple way to account for this effect is to select a number of typical water depths (e.g. 3 different water depths) and calculate the accumulated fatigue damage, η , for critical joints for each water depth. In accordance with the simplified fatigue evaluation described above the accumulated fatigue damage, D_i, for water depth i, may be expressed as:

$$D_i = \frac{1}{s} \left(\frac{\Delta \sigma_i}{\Delta \sigma_{o,i}} \right)^m$$
 where

 D_i = accumulated fatigue damage for water depth i.

s = total number of water depths considered.

 $\Delta \sigma_{\rm i}$ = the hot spot stress range with return period 20 years.

- $\Delta \sigma_{0,i}$ = the maximum allowable hot spot stress range, referred to a return period of 20 years.
- m = inverse slope of the S-N curve m = 3).

The total accumulated fatigue damage, D, may be calculated as: $D = \sum_{i=1}^{s} D_i$

7.9 Weibull parameter (h)

The Weibull parameter (h) determines the shape of the long term distribution as shown in Figure 7-2. The Weibull parameter depends on such factors as:

- Wave climate (long term distribution of wave heights)
- Water depths
- Type of structure (drag vs. inertia dominated load).
- Dynamic amplification
- Position of the fatigue location in the structure (lower leg/spudcan, splash zone, upper leg/hull structure, etc.)

The Weibull parameter may be corrected for areas where the wave height Weibull parameter is different from 1,0 as:

 $h = h_0 h_w$

where

 $h_0 = \text{stress Weibull parameter if } h_w = 1.0$

 h_w = wave height Weibull parameter being typically 1.0 for harsh environments, e.g. as North Sea 0.7-0.9 for calm areas subject to short periods of heavy weather (Monsoons, Typhoons etc.).



Figure 7-2 Long term stress distributions described by Weibull distributions.

In accordance with the previously described load and response analyses, it is assumed that the dynamic stress is related to the wave height like:

$$\Delta \sigma = (a_1 H + a_2 H^2)$$

where

H = wave height

 a_1, a_2 = stress coefficients, including dynamic effects.

The first term in this equation represents that portion of the dynamic stress which is proportional to the wave height (inertia forces and drag forces resulting from the wave/current interaction).

The second term in this equation represents that portion of the dynamic stress which is proportional to the square of wave height (drag forces).

It is recommended to determine a_1 and a_2 by adjusting the expression for $\Delta\sigma$ above to the actual function for $\Delta\sigma$, e.g. derived by analysing wave load and stresses for different values of H.

The ratio between the wave drag force induced stress and the total dynamic stress at maximum wave height is defined by:

$$a = \frac{H_{\text{max}}}{H_{\text{max}} + \frac{a_1}{a_2}}$$

By use of the average relation between wave height and wave period, given in 2.2, it is possible to determine the shape of the long term distribution. The result of a number of computations is shown in Figure 7-3. The parameter h is given as a function of load combination, a, and natural period, T_0 . The total damping ratio was assumed to be 8 percent, but the results were not very sensitive to variations between 5 and 10 percent. In many cases the calculated long term distributions could not be fitted by any Weibull distribution. In such cases h defines the Weibull distribution which would lead to the same fatigue damage prediction in combination with a typical S-N curve with m = 3. See also /25/ and /26/.



Figure 7-3 Weibull parameter h

8. Accidental strength (ALS)

8.1 General

In the safety evaluation of a platform concept, accidental loads should be identified and taken into consideration. For impact types of accidental load such considerations lead to recommendation of structures of a certain robustness or ductility or redundancy.

Requirements for Accidental Limit States (ALS) and structural redundancy of slender main load bearing structural elements are given in DNV-OS-C101 Sec.6.

The damaged condition may be divided in two main groups:

- Structural redundancy comprising fracture of bracing, chord or joint for legs
- The unit shall be designed for environmental loads with return period not less than 1 year after damage, see DNV-OS-C101 Sec.3 B100.

8.2 Ship impact

The present section describes only the impact from a ship collision. Such an impact load is most likely to cause local damage to one of the legs only, but the possibility for progressive collapse and overturning should also be considered.

The overall lateral stiffness of a jack-up platform is rather small, normally an order of magnitude smaller than the stiffness of a corresponding jacket structure.

Because a ship collision is a hypothetical event, it is allowable to use very simplified methods of analysis. A simple two-degree-of-freedom system may normally be considered appropriate, see Figure 8-1.



Figure 8-1 Model for impact analysis.

Under the assumption that $k_L >> k_G$ it is found that the maximum interaction force between ship and platform is:

$$P_L = k_L (\delta_S - \delta_P) = v \sqrt{\frac{m_S k_L}{1 + \frac{m_S}{m_P}}}$$

The maximum global load effect is:

$$P_G = k_G \delta_P = v \sqrt{\frac{m_S k_G}{1 + \frac{m_P}{m_S}}}$$

where

$$k_L = \frac{1}{\frac{1}{k_S} + \frac{1}{k_L}}$$
 = combined local stiffness of ship and leg.

k _S	= stiffness of ship.
k _P	= local stiffness of platform leg.
k _G	= overall lateral stiffness of platform.
m _P	= mass of platform.
m _S	= mass of ship including added mass.
v	= ship velocity before collision

The values of k_G and m_P are to be evaluated with due consideration of ship/platform configuration. If the platform is hit in such a way that the resulting platform displacement is a pure translation (no rotation) then:

 $k_G = n k_e$ $m_P = n m_e$

where n is the number of legs and k_e and m_e are defined in 4.4.3.

The value of k_p may generally be determined as shown in Figure 8-2. However, if stresses in chords or bracings are found to exceed the elastic limit before the lateral load reaches the maximum value, P_L , the calculations should be based on non-linear methods. Such methods should account for formation of plastic hinges and buckling of compression members. In addition the strength of tubular joints has to be checked according to the methods described in 6.3. If ductile energy absorption shall be possible, it is necessary that the joints do not fail due to excessive punching shear stresses.



Lateral load: P Lateral displacement: δ . Local stiffness: $k_P = P / \delta$

Displacement contributions:

- Shear and bending deflection of leg
- Bending of chord
- Compression of bracings
- Deformation of chord wall.

Figure 8-2 Local stiffness of platform leg.

The value of k_S represents the local stiffness of the ship. Because the ship plating is relatively thin, the deformation of the ship will always be inelastic. Typical force-indentation curves are shown in Figure 8-3 to Figure 8-5. For the purpose of these calculations the selected value of k_S should be an average value for the actual range of displacements.

The value of P_L will in the first place be decisive for the extent of local damage, while P_G will be decisive for the risk of overturning.





Recommended force indentation curve for broad side impact with infinitely stiff vertical circular cylinder of 1.5 m and 10.0 m diameter. Boat displacement 5000 tons.



Figure 8-4

Recommended force indentation curve for stern impact with infinitely stiff vertical circular cylinder of 1.5 m and 10.0 m diameter. Boat displacement 5000 tons.





8.3 Damaged structure

For a jack-up leg of sufficient robustness a credible accidental load as described in 8.2 will normally not lead to any significant damage. However, for slender lattice legs it may be found that one chord element is damaged and made inefficient. According to the Rules, a platform with a credible damage shall survive under environmental conditions corresponding to a return period of 1 year.

If one chord element in a triangular lattice leg, see Figure 4-1, is removed, a hinge will be formed. The leg will then be unable to transfer horizontal loads to the sea bottom, but it may still be able to carry axial loads. This means that horizontal loads have to be redistributed to the intact legs, which will then be subjected to increased loading. However, the environmental loads corresponding to a 1 year return period are significantly smaller than those corresponding to a 100 year return period. It is also possible to allow higher stresses in a damage condition than in a normal intact operating condition.

A simplified two-dimensional model is used in order to illustrate the residual strength of a jack-up platform after the formation of a hinge in one of the legs, see Figure 8-6.



Figure 8-6 Redistribution of forces after the formation of a hinge in one leg.

9. Overturning stability analyses

9.1 Introduction

A jack-up in an elevated condition is exposed to environmental loads (wind, waves and current) which contribute to a resulting overturning moment, M_o. The functional loads (gravity loads) contribute to a resulting stabilizing moment, M_s. Safety against overturning requires that:

The stabilizing moment is greater than the overturning moment.

— The foundation (sea bed) is stable.

The operating condition is characterized by the functional loads associated with the drilling operations. For a cantilever type jack-up the stabilizing moment may be relatively small.

The survival condition is characterized by such a distribution of functional loads that the stabilizing moment is as large as possible.

When the environmental loads increase to a certain level, it is necessary to change mode, from the operating condition to the survival condition. The change of mode may involve extensive and time consuming operations. It is therefore essential that the criteria and the procedures for change of mode are clearly stated.

9.2 Stabilizing moment

The stabilizing moment due to functional loads should be calculated with respect to the assumed axis of rotation.

For jack-ups with separate footings (spudcans) the axis of rotation may, in lieu of a detailed soil-structure interaction analysis, be assumed to be a horizontal axis intersecting the axis of two of the legs. It may further be assumed that the vertical position of the axis of rotation is located at a distance above the spudcan tip equivalent to the lesser of:

- a) half the maximum predicted penetration
- b) half the height of the spudcan.

(See Figure 9-1.)

For jack-up with mat support, the location of the axis of rotation is to be specially considered.

Any possible stabilizing effect of non-uniform soil reaction on separate leg footings should be neglected in the overturning stability analysis.

In lieu of other information, the stabilizing moment in 9.4 may be based on that 50% of the variable hull weight is included in the total elevated hull weight. This weight is to be included together with the most demanding rig global centre of gravity position.

For check of foundation in 9.5 also 100% of the variable hull weight should be considered in the stabilizing moment.

The stabilizing moment may be taken as:

$$M_s = M_{so} - \frac{nP(e_o + e)}{1 - \frac{P}{P_E}}$$

where

 M_{so} = stabilizing moment as calculated if the legs are perfectly straight and vertical.

n = number of legs. e_0 = maximum horizontal offset of the platform, see 4.4.7.

- = sideway 1st order deflection of barge due to wind, waves and current. e
- P = average axial leg load.
- $P_{\rm E}$ = Euler load on one leg, see in Appendix A.2.



Figure 9-1 Axis of rotation (A-A).

9.3 Overturning moment

The overturning moment due to wind, waves and current should be calculated with respect to the axis of rotation, defined in 9.2.

The calculation of overturning moment should account for the dynamic amplification of the combined wave/ current load effect.

If the calculation is based on a linear static response analysis, this effect may be approximately accounted for by use of amplification factors applied as follows:

$$M_o = M_{WD} + M_{W/C}$$

where

 M_{WD} = overturning moment due to wind.

 $M_{W/C}$ = overturning moment due to the combined effect of waves and current, incl. dynamic effects. See 4.4.3-0.

Any possible effect of non-uniform soil reaction on separate leg footings should be neglected in the overturning stability analysis. Consequently the legs should be considered pinned at the axis of rotation.

9.4 Design requirement

The design requirement with respect to overturning is:

 $M_s \ge \gamma M_o$

where γ is a safety factor defined in DNV-OS-C104 Sec.8.

The design requirement should be checked for the most unfavourable direction and combination of loads. It is normally assumed that wind, waves and current are coincident in direction.

9.5 Foundation stability

Problems concerning the foundation stability are related to design of legs and leg footings or mat, and soil conditions.

Legs with separate footings (spud cans) may penetrate the sea bed to a considerable depth, see 2.6. However, deep penetration represents in itself no hazard. The risk is the possibility for a sudden and unexpected penetration. The design philosophy is that the footings shall be preloaded to at least the same vertical load as the maximum combined effect of functional loads and extreme environmental loads.

- During preloading, a sudden penetration of one leg may lead to excessive tilt and damage to legs and jacking mechanisms. The presence of sand layers or crusts over soft clays is considered responsible for such accidents.
- During extreme storm conditions, the foundation will have to react to both vertical and horizontal loads. The bearing capacity of soil under combined horizontal and vertical loading may be significantly smaller than in the case of pure vertical loading as simulated by the preloading. Uncertainties regarding the bearing capacity are also due to dynamic loading being responsible for rocking of the footings, associated with nonuniform soil reaction, see Classification Note No. 30.4 /10/.

A mat foundation will normally have a very small penetration even in a soft clay. In stiff clays and sand, the unevenness of the sea bed may lead to non-uniform distribution of soil reaction, and the contact with the sea bed may be limited. In an extreme case, this may lead to rocking of the mat under dynamic loading.

For mat foundations, there are in principle two failure modes:

- Sliding of the mat over the surface of the sea bed or along a shallow failure surface directly under the skirts due to horizontal forces.
- Overturning due to insufficient bearing capacity under the most stressed edge of the mat caused by horizontal forces and overturning moment combined with the weight of the jack-up and the wave pressure on the sea bed.

The effect of repeated loading on sands and clays may be significant and lead to a strength reduction of order 30-40% for clay and even more for loose sand /27/.

10. Air gap

10.1 Introduction

The air gap is defined as the clear distance between the hull structure and the maximum wave crest elevation.

The crest elevation above the still water level is defined in Figure 10-1, and may be calculated according to Figure 10-2.

The still water level (SWL) is defined as the highest astronomical tide including storm surge, see 2.5.

10.2 Requirement

The air gap is not to be less than 10 per cent of the combined astronomical tide, storm surge and wave crest elevation above the mean water level (MWL), see 2.5.

However, the air gap is not required to be greater than 1.2 m, see Figure 10-1.

10.3 Caution

For deep water locations the air gap requirement is vital for the evaluation of the jack-up's suitability. In this connection it is very important that the leg penetrations are predictable, see Classification Note No. 30.4 /10/.



MWL = mean water level. SWL = still water level. CE = wave crest elevation above SWL.

SE = surface elevation above SWL.

Figure 10-1 Definition of air gap.



 η_o = crest elevation above still water (metres). H = wave height (metres). T = wave period (seconds).

H = still water depth (metres).



11. References

/1/ DNV-OS-C101 Design of Offshore Steel Structures, General (LRFD method), Det norske Veritas

/2/ DNV-OS-C104 Structural Design of Self-Elevating units (LRFD Method), Det norske Veritas

/3/ DNV-OS-C201 Structural Design of Offshore Units (WSD method), Det norske Veritas

/4/ DNV-RP-C201 Buckling Strength of Plated Structures, Det norske Veritas.

/5/ DNV-RP-C203 Fatigue Design of Offshore Steel Structures, Det norske Veritas.

/6/ DNV-OS-C502 Offshore Concrete Structures, Det Norske Veritas.

/7/ DNV-OS-J101 Design of Offshore Wind Turbine structures, Det Norske Veritas

/8/ DNV-RP-C205 Environmental Conditions and Environmental Loads, Det norske Veritas

/9/ Classification Note 30.1 Buckling Strength Analysis of Bars and Frames, and Spherical Shells, Sec.2 Bars and Frames, Det norske Veritas

/10/ Classification Note No. 30.4 "Foundation", Det norske Veritas.

/11/ Carlsen C.A., Kjeøy H.: "Structural behaviour of harsh environmental jack-ups", RINA London 25_27 Sept. 1985. Paper Series No. 85 P016, Det norske Veritas, 1985.

/12/ Arnesen K., Kjeøy H.: "Soil_structure interaction aspects for jack-up platforms", BOSS '88, NTH, Trondheim, June 1988. Paper Series No. 88 P017, Det norske Veritas, 1988.

/13/ Kjeøy H., Bøe N.G., Hysing T.: "Extreme response analysis of jack-up platforms", 2nd International Conference _ City University, London, 26_27 Sept. 1989. Paper Series No. 89 P012, Det norske Veritas, 1989.

/14/ Carlsen C.A., Gundersen H.B., Gran S.: "Environmental data in operation and design case _ mobile rigs", Paper Series No. 18 P017, Det norske Veritas, 1981.

/15/ Hallan M.G., Heat N.J., Wotton L.R.: "Dynamics of marine structures", CIRIA Underwater Engineering Group, Report UR8, 1977.

/16/ Nordenstrøm N.: "A method to predict long-term distributions of waves and wave-induced motions and loads on ships and other floating structures", Det norske Veritas, Publication No. 18, 1973.

/17/ Odland J.: "Response and strength analysis of jack-up platforms", Norwegian Maritime Research, Vol. 10, No. 4, 1982.

/18/ Tellsgård F.: "Wave-induced motion responses of a jack-up rig in transit condition", Note No. CP4-107-81, Computas, 1981.

/19/ Endley S.N., Rapport V., Thompson P.J.: "Prediction of jack-up rig footing penetration", OTC 4144, 1981. /20/ Hattori Y. et.al.: "Full-scale measurement of natural frequency and damping ratio of jack-up rigs and some theoretical considerations", OTC 4287, 1982.

/21/ Structural Stability Research Council: "Guide to Stability Design Criteria for Metal Structures", Editor: B.G. Johnston, John Wiley and Sons, 1976.

/22/ "Alexander L. Kielland-ulykken", NOU 1981: 11, Universitetsforlaget, Oslo, 1981.

/23/ Gran S.: "Fatigue and Crack Failure Prediction Methods in Marine Structures", Det norske Veritas, Report No. 80-0688, 1980.

/24/ Gran S.: "Evaluation of high cycle fatigue in welded steel connections", Det norske Veritas, Report No. 76-339, 1977.

/25/ Marshall P.W., Luyties W.H.: "Allowable stresses for fatigue design", BOSS Conference, 1982.

/26/ Vandiver J.K.: "The sensitivity of fatigue life estimates to variation in structural natural periods, modal directional spreading of the seas", BOSS Conference, 1982.

/27/ Kvalstad T.J., Dahlberg R.: "Cyclic Behaviour of Clay as Measured in Laboratory", Int. Symp. on Soils under Cyclic and Transient Loading, Swansea, 1980.

/28/ Sarpkaya T., Isaacson M.: "Mechanics of wave forces and offshore structures", 1981.

APPENDIX A SIMPLIFIED GLOBAL LEG MODEL OF LATTICE LEGS

A.1 Equivalent stiffness for Lattice legs

The key parameter for the evaluation of the equivalent cylindrical leg is the "equivalent shear area" of a twodimensional lattice structure, see Table A-1.

Stiffness parameters for legs with three or four chords are given in Table A-2. An important feature of such legs is that the stiffness is the same for all orthogonal directions. However, this is not the case for legs with racks on only two of the chords.

In practical cases it may be difficult to represent the complete leg by one beam element. This is because the stiffness normally varies over the length. The stiffness is closely related to the steel weight, and a typical steel weight distribution of a jack-up leg is shown in Figure A-2.

The overall stiffness or flexibility of jack-up legs also depends on the boundary conditions. A simplified model which accounts for the most important effects is shown in Figure A-3. This model is subjected to a lateral load, P. The corresponding shear force and bending moment distributions are shown in Figure A-3. The lateral overall stiffness is given by:

$$k = \frac{1}{f_B + f_Q}$$

where

 f_B = bending flexibility.

 f_{O} = shear flexibility.

With reference to Figure A-3, f_B and f_O may be taken as:

$$f_B = \frac{l^3}{3EI} \left[1 - \frac{3}{2} \frac{\mu}{1 + \mu} + \frac{i}{1 + \mu} \frac{d}{l} \right]$$
$$f_Q = \frac{l}{GA_Q} \left[1 + \frac{a}{1 + \mu} \frac{l}{d} \right]$$

Where

$$a = \frac{A_Q}{A_{QO}} \left[1 - \beta \right]$$

$$i = \frac{I}{I_o} \left[1 - \beta \left(1 - 3\frac{b}{d} + \frac{3}{2} \left(\frac{b}{d} \right)^2 \right) \right]$$

- A_{QO} , I_{O} = average shear area and moment of inertia of the portion of the leg located between upper and lower guides.
- A_Q , I = average shear area and moment of inertia of the portion of the leg which is below the lower guides. β = coefficient which determines the fraction of the leg bending moment which is reacted by vertical forces.
- μ = coefficient which determines the leg bending moment at the bottom.

The coefficients β and μ may be determined from:

$$\beta = \frac{1}{1 + G A_{QO} \frac{d}{k_I}}$$

$$\mu = \frac{1 + \frac{2}{3}i\frac{d}{l} + \frac{2 a E I}{l d G A_{Q}}}{1 + \frac{2 E I}{k_{s} l}}$$

where

 k_J = rotational spring stiffness of the jacking mechanisms, see A.3. k_s = rotational spring stiffness at the bottom, see Classification Note No. 30.4, /10/. Alternatively the overall lateral stiffness may be expressed by:

$$k = \frac{3EI}{cl^3}$$

where (see Figure A-1):

$$c = 1 - \frac{3}{2} \frac{\mu}{1 + \mu} + \frac{i}{1 + \mu} \frac{d}{l} + \frac{3EI}{l^2 GA_o} \left(1 + \frac{a}{1 + \mu} \frac{l}{d} \right)$$



Figure A-1 The stiffness coefficient c

Table A-1 Equivalent shear area for two-dimensional lattice structures .			
	Structure	Equivalent shear area	
a	An Ay h	$A_{Qi} = \frac{(1+\nu)sh^2}{\frac{d^3}{2A_D} + \frac{s^3}{6A_C}}$	
b	A A A A A A A A A A A A A A A A A A A	$A_{Qi} = \frac{(1+\nu)sh^{2}}{\frac{d^{3}}{A_{D}} + \frac{h^{3}}{8A_{V}} - \frac{s^{3}}{NA_{C}} \left(\frac{N^{3}}{3} - \sum_{i=1}^{N} i^{2}\right)}$ N = No of active bays	
c	An h	$A_{Qi} = \frac{(1+\nu)sh^2}{\frac{d^3}{4A_D} - \frac{s^3}{12A_C}}$	
d	AD AD AD	$A_{Qi} = \frac{(1+\nu)sh^2}{\frac{d^3}{2A_D} + \frac{h^3}{2A_V} + \frac{s^3}{6A_C}}$	
e		$A_{Qi} = \frac{48(1+\nu)I_G}{s^2 \left(1 + \frac{d}{s} \frac{I_G}{I_B}\right)} \left(\frac{d}{h}\right)$	
Table A-2 Equivalent section properties of three-dimensional lattice legs.			
--	-----------------	---	
Leg type		Equivalent properties	
a)	Aci Aci Y	$A = 3A_{Ci}$ $A_{Qy} = A_{Qz} = \frac{3}{2}A_{Qi}$ $I_y = I_z = \frac{1}{2}A_{Ci}h^2$ $I_T = \frac{1}{4}A_{Qi}h^2$	
b)	h h h	$A = 4A_{Ci}$ $A_{Qy} = A_{Qz} = 2A_{Qi}$ $I_{y} = I_{z} = A_{Ci}h^{2}$ $I_{T} = A_{Qi}h^{2}$	
c)	Azi Agi	$A = 4A_{Ci}$ $A_{Qy} = A_{Qz} = 2A_{Qi}$ $I_{y} = I_{z} = A_{Ci}h^{2}$ $I_{T} = A_{Qi}h^{2}$	



Figure A-2 Typical steel weight distribution of a leg.



Figure A-3 Leg stiffness model

A.2 Euler load of legs

The Euler load of one leg is given by:

$$P_{\rm E} = \pi^2 {\rm EI}/\left({\rm Kl}^2\right)$$

where K is an effective length factor which may be taken as: $K = 2\sqrt{c}$, and c is the coefficient defined in A1.

A.3 Leg-to-hull interaction

In principal the leg bending moment is reacted partly by horizontal forces from the guides and partly by vertical forces from the jacking mechanisms. If a fixation system is present and engaged, almost the entire leg bending moment will be transferred by vertical forces at the fixation system.

The relative distribution between horizontal and vertical forces may be defined by a factor β :

 $P = \frac{\text{Part of bending moment reacted by vertical forces}}{\text{Total bending moment}}$

The value of beta depends primarily on how the jacking mechanisms are supported by the hull (fixed or floating type) and whether a fixation chocking system is present and engaged.



M = bending moment.

Q = shear force.

Figure A-4 Leg-hull interaction.

The most important effects of the coefficient β are:

- Shear forces between upper and lower guides is highly dependent on the β -factor, see Figure A-3 and Figure A-4. This shear force is very important, in particular for lattice legs, where large overall shear forces produce large axial forces in the brace members. In addition, large bending stresses will be induced in the chord due to the guide reactions, in particular when the guides are positioned between bay levels, see Figure 5-4.
- The highest natural period will be affected by the value of β . See equations given in 4.2.

A.3.1 Fixation system

Due to the rigidity of the fixation system, the majority of the leg bending moment will be resisted by the fixation system and only a small portion of the leg bending moment will be resisted by the guide structures. Typical β -values for this system will be in the order of $0.8 < \beta < 1.0$.

The following is to be considered when modelling of units with a fixation system:

- The leg-to-hull connection can be modelled simply by using vertical reactions in the chord members, at the level of the fixation system (β assumed = 1.0).
- The jacking system should be capable of carrying safely the gravity loads plus any environmental loads arising when the fixation system is not engaged.

A.3.2 Jacking system (pinions)

A *fixed jacking system* is one which is rigidly mounted to the jack house and hence to the hull. Such a stiff system tends to absorb most of the bending moment in the leg as well as the gravity loads. Due to some torsional flexibility in the pinion gear train, and the flexibility of the leg, the guides come into contact with the legs and there is some transfer of overturning moment as a horizontal couple at the guides. Typical β -values for this system will be in the order of $0.4 < \beta < 0.8$

The following is to be considered when modelling of units with a fixed jacking system:

- Establish the value or range of β from the relevant stiffness, i.e. the effective vertical stiffness of the pinions, versus the stiffness of the leg under shear displacements, see A.3.3.
- If the pinions are unopposed, see Figure 5-5, the pinion loads will introduce a resultant horizontal force and associated bending stresses in the chord members and axial loads in the braces, in addition to vertical loads in the chord. These stresses should be accounted for.
- If the pinions are opposed, see Figure 5-5, no bending stresses are induced in the chords at the pinion-rack interface.
- The most critical location for chord stresses is likely to be at the level of the lower guide because the leg bending moment is at maximum at this location. The local chord bending stresses caused by the lower guide reaction should be accounted for. The guide reactions should be considered to act (i) at the mid-span of the chord and (ii) opposite a node, in order to capture all critical situations. The lower guide reaction should be assumed to be distributed over the guide length in a realistic manner, see Figure 5-4.

— The holding capacity of the jacking mechanism should be sufficient to carry the gravity and environmentally induced load components plus a degree of safety as given in the Rules.

A *floating jacking system* is one which is mounted to the jack house via flexible upper and lower shock pads. Under environmental loading such a flexible system rotates and the guides come into contact with legs and resist a considerable proportion of the leg bending moment.

In the extreme case of deep flexible rubber pads the guides will be required to resist the entire leg bending moment and the jacking mechanism will carry only the gravity loads.

Typical β -values for this system will be in the order of $0 < \beta < 0.4$

For a floating jacking system a major part of the leg bending moment will be carried by horizontal forces in the upper and lower guides leading to high shear forces in this portion of the leg. The large guide reactions will also be decisive for the design of the jack house.

The modelling considerations for a unit with floating jacking system will be as described for the fixed jacking system (see above), taking into account the effects of a further reduction in the β -values.

A.3.3 Leg-to-hull interaction based on equivalent leg beam model

For analysis of the global response it may normally be assumed that the stiffness of the hull structure is infinitely high compared to the stiffness of the legs. The jacking mechanisms may be represented by a rotational spring. If this rotational spring stiffness is k_J , the coefficient β is approximately given by:

$$\beta = \frac{1}{1 + d G \frac{A_Q}{k_L}}$$

where

 A_Q = shear area of leg, see Table A-1 and Table A-2.

d = distance between upper and lower guides.

For illustration it is shown in Figure A-5 how the rotational spring stiffness may be determined for a simplified two-dimensional structure.



 $k_j = 1/2 \ kh^2$

Figure A-5 The spring coefficient k_J

In the case of resilient mounted jacking mechanisms it should be noticed that only compressive forces can be transferred through the shock pads. This means that β can never exceed the value:

$$\beta = \frac{M_c}{M_o}$$

where

 M_0 = maximum leg bending moment.

 M_{c}° = bending moment at which one of the vertical reaction forces vanishes.

For the case shown in Figure A-5:

$$M_c = \frac{P d}{2}$$

A.4 Natural period for simplified Jack-up models

For simplified models of a jack-up, the procedure below may be followed. The natural period is the inverse value of the natural frequency:

$$T_{o} = \frac{1}{f} = 2\pi \sqrt{\frac{m_{e}}{k_{e}}}$$

where

f = natural frequency.

 $k_e = effective stiffness of one leg.$

 m_e = effective mass related to one leg.

The effective stiffness depends on several parameters:

Leg stiffness

— Leg/hull interaction

- Leg/bottom interaction.

For the elevated condition the effective stiffness may be taken as:

$$\mathbf{k}_{e} = \left(1 - \frac{\mathbf{P}}{\mathbf{P}_{E}}\right)\mathbf{k}$$

where

k = lateral overall stiffness as defined in 4.2.2.

P = axial force in one leg due to the functional loads.

 P_E = Euler load as defined in A.2.

The effective mass for the elevated condition may be taken as:

$$m_e = c_1 M_H + c_2 M_L$$

where

- $M_{\rm H}$ = total mass of the hull with all equipment and the portions of the legs located above the lower guides or fixation system.
- M_L = mass of the portion of one leg located between the lower guides or fixation system and the top of the spud cans (added mass included).

c =
$$\frac{1}{n}$$
 for modes 1 and 2.
= $\frac{1}{n} \left(\frac{r_o}{r}\right)^2$ for mode 3.
n = number of legs.

 $c_2 = 0.5 - 0.125 \,\mu$

 r^2 = distance from the hull's centre of gravity to the centre of the legs.

 $r_0 = radius of gyration of the mass M_H with respect to the vertical axis through the centre of gravity.$

 μ = bottom restraint parameter, see A.1.

Because r_0 is normally smaller than r, the natural frequency corresponding to mode 3 is normally higher than the natural frequency corresponding to modes 1 and 2.

A.5 Simplified damping at resonance

The value of the dynamic amplification factor at resonance is governed by the damping ratio, ξ .

$$DAF(r) = \frac{1}{2\xi}$$

In order to account for the effect of irregular sea, a "stochastic dynamic amplification factor" is defined:

$$SDAF = \frac{1}{2\xi_{ST}}$$

where ξ_{ST} is an apparent damping ratio.

In this connection it may be assumed that the transfer function at resonance peak may be written as:

$$TR = C * DAF$$

where C is a constant. Consequently:

$$SDAF^{2} = \frac{\Sigma C^{2} DAF^{2} S(\omega) d\omega}{\Sigma C^{2} S(\omega) d\omega}$$

where S (ω) is the wave energy spectrum.

A parametric study based on the Pierson-Moskowitz wave spectrum shows that the SDAF has a maximum when the peak of the wave spectrum is approximately equal to the natural period. The SDAF was found to be rather insensitive to variations in the natural period for the range 5-10 seconds. The following approximate relation was obtained:

$$\xi_{\rm ST} = 0.75 \ \xi^{0.65}$$

For the most interesting range:

$$\xi_{\rm ST} \sim 2 \xi$$

This result is exclusive of the effect of short-crested ness in 2.2. The ξ_{ST} may also be used for other values of T.

A.6 Idealization of lattice legs for wave, current and wind loads

Hydrodynamic loads on a lattice leg may be obtained either from a direct analysis of the complete structure or from a simplified analysis of an "equivalent" cylindrical leg. For a simplified analysis it is necessary to determine the equivalent cylinder diameter and the corresponding hydrodynamic coefficients.

The equivalent inertia coefficient may be chosen as: $C_{IE} = 2.0$

The equivalent diameter is then determined by:

$$D_E = \sqrt{\frac{4 \, V_B}{\pi \, s}}$$

where,

 $V_{\rm B} = \Sigma A_{\rm i} l_{\rm i}$ - total volume of one bay.

- A_i^{-} = cross sectional area of member i.
- $l_i = length of member i.$ s = length of one bay
- Σ = indication of summation over all members in one bay.

The equivalent drag coefficient is given by:

$$C_{DE} = \sum C_{DEi}$$

$$C_{DEi} = \left(\sin^2\beta + \cos^2\beta\sin^2\alpha\right)^{3/2} C_{Di} \frac{D_i l_i}{D_E s}$$

where.

- $C_{Di} = drag$ coefficient of member i.
- D_E^{DT} = equivalent diameter of leg.
- D_i^- = diameter of member i.
- $l_i = length of member i.$
- = length of one bay. S
- = angle which determines the flow direction, see Figure A-6. β = angle which determines the inclination α of diagonal members, see Figure A-6.
- = indicates summation over all members in one bay. Σ

The expression for C_{DEi} may be simplified for vertical and horizontal members.

Vertical members (chords):

$$C_{DEi} = C_{Di} \frac{D_i}{D_E}$$

Horizontal members:

$$C_{DEi} = \sin^3 \alpha \ C_{Di} \frac{D_i l_i}{D_E s}$$



Figure A-6 Flow through a lattice leg

A.7 Forces in chord and brace members from equivalent leg section

For the analysis of a portion of a lattice leg, it is normally allowable to neglect the effect of loads distributed over the length of the individual members (other than guide and jacking machinery reactions). This is illustrated by example shown in Figure A-7.

Except for the specific areas where large forces are introduced, the distribution of local forces and moments may be obtained by use of simple methods.

The overall axial forces and leg bending moments are reacted by axial forces in the chords. If all chords are identical, the required cross sectional properties may be obtained from Table A-1 and Table A-2.

Shear forces and torsional moments are reacted by axial forces in the bracing members. An overall shear force with a given direction is distributed as shown in Figure A-8. An overall torsional moment is uniformly distributed around the circumference. The resulting shear force acting in a plane through two of the chords is referred to as Qi. The axial forces in the bracing members may then be obtained from Figure A-9 for two commonly used bracing systems.



Maximum bending stress σ .

Selfweight :
$$\sigma = \frac{\gamma_s L^2}{3 D}$$

Buoyancy : $\sigma = \frac{\gamma L^2}{12 t}$
Dragforce : $\sigma = \frac{\gamma C_D H L^2}{84 Dt}$

(Assumed wave steepness = 1/7)

D = diameter.

- L = length.
- T = wall thickness.
- H = wave height.
- $C_D = drag \text{ coefficient.}$
- γ = specific weight for sea water.
- $\gamma_{\rm s}$ = specific weight for steel.

Example:

L = 5 m. D = 0.3 m. H = 20 m. CD = 1.0. t = 30 mm.

- Self weight: σ = 2.2 N/mm²
- Buoyancy: $\sigma = 0.7 \text{ N/mm}^2$
- Drag force: $\sigma = 6.8 \text{ N/mm}^2$.

Figure A-7 Effect of distributed loads on individual members in lattice legs.



Figure A-8 "Shear flow" in lattice legs.



INDICES: D = DIAGONAL, H = HORIZONTAL

Figure A-9 Axial forces in bracing members.

A.8 Leg Joint eccentricity

Members at a joint may be so designed that their axes do not intersect at one point. This represents an eccentricity which causes a bending action due to the eccentricity moment M_e, see Figure 5-6.

According to linear elastic theory this moment is shared between the members according to their relative stiffness. The moment acting at the end of member (i) is therefore given by:

$$M_i = M_e \frac{I_i / l_i}{\sum I_j / l_j}$$

where Σ indicates summation over all members in the joint. If considered necessary, second order effects may be taken into account by a reduction of stiffness of the compression members like:

$$I'_{i} = I_{i} \left(1 - \frac{P_{i}}{P_{Ei}} \right)$$

where P_i is the axial compressive load, and P_{Ei} is the Euler load of member (i).

The overall shear force in the leg obtained by 1st order elastic theory should be corrected for 2nd order bending effect as:

$$Q' = Q \frac{\pi}{2} \frac{r}{P_E - P} \cos \frac{\pi x}{2 \,\mathrm{h}}$$

which is maximum where M is zero (x = 0 where M is zero).

A.9 Simplified analysis of legs in Transit

The approach as described in 4.5.3 can also be used with equivalent leg stiffness model as described below. The effect of heave, surge and sway are implicitly accounted for by use of a specified load scaling factor = 1.2. For calculation of leg forces it is assumed that the roll or pitch motion can be described by:

$$\theta = \theta_0 \sin \frac{2 \pi t}{T_0}$$

where

= time variable. $T_o =$ natural period of roll or pitch. $\theta_o =$ amplitude of roll or pitch.

The axis of rotation is assumed to be located in the water plane, see Figure A-10.

The acceleration of a concentrated mass located at a distance r from the axis of rotation is then:

$$a = -\varepsilon_0 r \sin \frac{2\pi t}{T_0}$$

Where

$$\varepsilon_o = \left(\frac{2 \pi}{T_o}\right)^2 \theta_c$$

The natural period is the inverse value of the natural frequency:

$$T_{o} = \frac{1}{f} = 2\pi \sqrt{\frac{m_{e}}{k_{e}}}$$

where

f = natural frequency. $k_e = effective stiffness of one leg.$ $m_e = effective mass related to one leg.$

For the transit condition the lowest natural frequencies correspond to bending deflection of each individual leg cantilevered upwards from the hull jacking structure

For the transit condition the effective stiffness may be taken as:

 $k_e = k$

where k is the lateral overall stiffness as defined in A.1 with $\mu = 0$, and b equal to the distance between upper guides and the centre of the jacking mechanisms (or to the locking system, if such a system is engaged).

If the mass of the leg is uniformly distributed, the effective mass for the transit condition may be taken as /15/:

$$m = 0.24 M_{I}$$

where

 M_L = mass of the portion of one leg located above the upper guides.



Figure A-10 Motion of a jack-up platform in transit.

The maximum forces per unit length of the leg, at a position defined by the coordinate x, see Figure A-10, are:

a) Lateral forces

Static force: $F_{TS} = m(x) g \sin \theta_0$ Inertia force: $F_{TD} = m(x) \varepsilon_0 x$ Wind force: $F_W = 0.5 \rho C_D D [v(x) \cos \theta_0]^2$

b) Axial forces

Static force: $F_{LS} = m(x) g \cos \theta_0$ Inertia force: $F_{LD} = m(x) \varepsilon_0 d$

where

m(x) = mass per unit length of the leg at the position x.

- v(x) = wind velocity at the position x.
- g = acceleration of gravity.
- ρ = density of liquid.
- D = cross sectional dimension perpendicular to the flow direction (for a circular cylindrical leg, D is the diameter).

 C_D = drag (shape) coefficient.

The leg bending moment, shear force and axial force may be obtained in a straightforward way by applying these distributed loads to a structural model of the leg, ported in the hull. However, since only the maximum leg section forces and moments at the intersection with the hull are of real interest, the required information may be obtained by integration of the load intensities over the leg length.

By assuming that the leg mass is uniformly distributed over the length, and that the wind profile is defined according to 2.4, the following explicit expressions are obtained:

a) Lateral forces

 $F_{TS} = M_L g \sin \theta_0$, acting at:

 $F_{TD} = M_L \varepsilon_0 l (0.5 + b/l)$, acting at:

$$x_D = \frac{2}{3} l \frac{1 + 3(b/l) + 3(b/l)^2}{1 + 2(b/l)}$$

 $x_{\rm S} = l (0.5 + b/l)$

 $F_W = 0.5 \text{ c} \rho C_D D v_R^2 z_o$, acting at:

$$x_{W} = \frac{z_{W} - z_{L}}{\cos \theta_{o}} + b$$

b) Axial forces

$$F_{LS} = M_L g \cos \theta_0$$

$$F_{LD} = M_L \varepsilon_0 d$$

c = 0.85 cos
$$\theta_0$$
 [$(z_H/z_0)^{1.18} - (z_L/z_0)^{1.18}$]

$$z_{W} = 0.54 z_{o} \frac{(z_{H} / z_{o})^{2.18} - (z_{L} / z_{o})^{2.18}}{(z_{H} / z_{o})^{1.18} - (z_{L} / z_{o})^{1.18}}$$

- $z_{\rm H}$ = vertical distance from the still water level to the top of the leg.
- z = vertical distance from the still water level to lower exposed point of the leg (at upper guides).
- z_0 , v_R = reference parameters for wind velocity, see 2.4.

 M_L = total mass of that portion of the leg which is located above upper guides.

b = as in Figure A-10.

When the bending moment (M_o) and the shear force (Q_o) at the upper guides have been determined, the distributions of bending moment and shear force may be established according to Figure A-11.

The coefficient β which determines the fraction of the bending moment reacted by vertical forces from the jacking mechanisms is explained in A.3.



Reactions:

Lower guides, $R_L = (1-\beta) M_o/d$ Upper guides, $R_U = R_L + Q_o$

Figure A-11 Bending moment and shear force distributions in transit.

A.10 Simplified Wave motion analysis

In lieu of a rigorous wave motion analysis, the natural period for the roll or pitch motion may be calculated from:

$$T_o = 2\pi \sqrt{\frac{r_o^2 - a_o^2}{g \; GM}}$$

where

 r_0 = radius of gyration for roll or pitch with respect to an axis located in the water plane.

 a_0 = vertical distance between the water surface and the true axis of roll or pitch.

GM= transverse or longitudinal metacentric height.

The distance a₀ may conservatively be taken as the vertical distance between the water surface and the centre of gravity. Short periods give largest accelerations.

The radius of gyration, r_0 , may be taken as:

$$r_{O} = \sqrt{\frac{I_{m}}{M_{m}}}$$

where

 $I_m = I_L + I_H + I_A$ = mass moment of inertia with respect to axis of roll and pitch.

 $M_m = n M_L + M_H = mass.$

n = number of legs.

 I_L = mass moment of inertia of legs.

 $I_{\rm H}$ = mass moment of inertia of hull.

 I_A = added mass moment of inertia.

 M_L = mass of one leg.

 $M_{\rm H}$ = mass of hull.

For approximate analysis the mass may be assumed to be uniformly distributed, and the various contributions to the total mass moment of inertia may be determined in accordance with Table A-3 and Figure A-12. The formulae should only be used if $0.5 < C_W < 1.0$, where C_W is waterplane area coefficient.



$$C_W = \frac{A_W}{L B}$$

 A_W = water plane area

Figure A-12 Jack-up platform in transit.

Table A-3 Mass moment of inertia		
Roll		
I _L =	$l^2 M_L \sum_{i=1}^n \left[\frac{1}{12} + \left(\frac{d_i}{l} \right)^2 \right]$	
$I_{\rm H} =$	$0.042 (3 - 1/C_W) M_H B^2$	
$I_A =$	$0.007 \frac{B}{d} (4 - 1.5 / C_W) M_m B^2$	
Pitch		
I _L =	$l^2 M_L \sum_{i=1}^n \left[\frac{1}{12} + \left(\frac{d_i}{l} \right)^2 \right]$	
$I_{H} =$	$0.028 (4 - 1/C_W) M_H L^2$	
I _A =	$0.009 \frac{L}{d} (3 - 1/C_W) M_m L^2$	