RECOMMENDED PRACTICE
DNV-RP-C103

COLUMN-STABILISED UNITS

FEBRUARY 2005

DET NORSKE VERITAS

FOREWORD

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Amendments and Corrections

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Main changes

— Text and definitions have generally been co-ordinated with the 2004 revisions of the relevant structural standards (e.g. references, terminology, definitions, lay-out of text, etc.)

— The text relating to 'tank pressures' (sec.3.8) has been amended. Formulations have been simplified and clarified.

— The text relating to support of mooring equipment (sec.6.1) has been updated and clarified.

CONTENTS

1. The Column-Stabilised Unit

1.1 Introduction

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

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

— DNV-OS-C101 Design of Offshore Steel Structures, General (LRFD method)
— DNV-OS-C103 Structural Design of Column-stabilised Units (LRFD method)
— DNV-OS-C201 Design of Offshore Units (WSD method).

LRFD is use of the load and resistance factor design method. WSD is use of the working stress design method.

Details given in this RP are referring to the LRFD method.

The design principles with use of the LRFD method are described in DNV-OS-C101 Sec.2 with different limit states. A limit state is a condition beyond which a structure or part of a structure exceeds a specified design requirement. 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 principles described in this RP may also be applied with use of the WSD method. The design principles with use of the WSD method are described in DNV-OS-C201 Sec.2, with design conditions described by different modes of operation (draughts) or phases during the life of the unit. The following design conditions shall normally be considered: installation, operating, survival, transit, accidental, and damaged.

1.2 Important concept differences

The methods outlined in this RP are mainly developed for the analyses of twin pontoon units and ring pontoon units. Consequently this should be taken into account when other concepts are considered.

Ring pontoon designs normally have one continuous lower hull (pontoons and nodes) supporting 4-8 vertical columns. The vertical columns are supporting the upper hull (deck).

Twin pontoon designs normally have two lower hulls (pontoons), each supporting 2-4 vertical columns. The 4-8 vertical columns are supporting the upper hull (deck). In addition the unit may be strengthened with diagonal braces supporting the deck and horizontal braces connecting the pontoons or columns.

There are basically two ways of keeping the unit in position:

— mooring by anchor lines (passive mooring system)
— dynamic positioning by thrusters (active mooring system).

A combination of these methods may also be utilised.

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

— production
— drilling
— accommodation
— special services (e.g. diving support vessel, general service, pipe laying vessel, etc.).

Units intended to follow normal inspection requirements according to class requirements, i.e. typically drilling units with inspection in sheltered waters or dry dock every 4-5 years, shall be designed with the design fatigue life equal to the service life, minimum 20 years, as given in DNV-OS-C103 Sec.5.

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-C103 Appendix A. These supplementary requirements for permanently installed units are related to:

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

1.3 Design principles and parameters

1.3.1 General

Structures and elements thereof, 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.

1.3.2 Design conditions

Different modes of operation of a column-stabilised unit are usually characterised in terms of “design conditions.” Changes in the design conditions of a column-stabilised unit are usually accompanied by significant changes in draught, ballast, riser connections, mooring line tension, or distance from an adjacent platform, etc. Limited variation of some of these parameters may be contained within a specific design condition, so the definition of design conditions is to some extent an arbitrary choice by the designer, arranged to cover all relevant combinations in a systematic and convenient way.

A typical set of design conditions is listed in Table 1-1. All relevant design criteria must be checked and satisfied for each design condition. The design criteria are expressed in terms of limit states.

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

1.3.3 Limit states

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	x	x	x	x		
FLS	(x)	x	(x)	(x)		
ALS					x	x

Limiting design criteria for transfer from one mode of operation (draughts) to another mode of operation shall be clearly established and documented. Different modes of operation or phases during the life of a column-stabilised unit may be decisive for the design. For each of the relevant design conditions the design criteria shall include relevant consideration of the following items:

— intact condition, structural response and strength (ULS, FLS, ALS)
— damaged condition, structural response and strength (ALS)
— air gap (ULS)
— compartmentation and stability requirements (intact (ULS) and damaged (ALS)).

1.4 Abbreviations

ALS	Accidental Limit States
DFF	Design Fatigue Factor
DNV	Det Norske Veritas
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
WSD	Working Stress Design

2. Environmental Conditions and Loads

2.1 Introduction

The suitability of a column-stabilised unit is dependent on the environmental conditions in the area of the intended operation. A drilling unit may be intended for worldwide operation or operation in a specific region. A production unit may be planned to operate at a specific site. Such a site may be harsh environment or benign waters.

Hence the environmental conditions and environmental loads depend on the area where the unit is intended to operate. A column-stabilised unit is normally designed for one of the following conditions:

— worldwide
— specific region or site(s).

The environmental conditions with general importance for column-stabilised units are described by a set of parameters for definition of:

— waves
— current
— wind
— snow and ice
— temperature
— water depth.

The applied environmental conditions should be stated in the design basis/design brief.

Typical environmental loads to be considered in the structural design of a column-stabilised unit are:

— wave loads, including variable pressure, inertia, wave "run-up", and slamming loads
— wind loads
— current loads
— snow and ice loads.

Due consideration should be made to site specific environmental phenomena such as hurricanes, cyclones etc.

Design for *worldwide operation* shall be based on Classification Note 30.5 with use of the scatter diagram for the North Atlantic as given in Classification Note 30.5 Table 3.3. See also 2.2 and DNV-OS-C101 Sec.3 E200, DNV-OS-C103 Sec.4 B100, DNV-OS-C103 Sec.5 B500 and DNV-OS-C103 Appendix B.

Design for *specific region or site* shall be based on specified environmental data for the area(s) the unit shall operate, see DNV-OS-C101 Sec.3 E and DNV-OS-C103 Appendix A and DNV-OS-C103 Appendix B. The parameters describing the environmental conditions shall be based on observations from or in the vicinity of the relevant location and on general knowledge about the environmental conditions in the area. Classification Note 30.5 may be used as guidance for determination of the environmental loads.

2.2 Environmental conditions

2.2.1 General

The most significant environmental loads for the hulls of column-stabilised units are normally those induced by waves. In order to establish the characteristic response, the characteristics of waves have to be described in detail. This description may either be based on deterministic design wave methods or on stochastic methods applying wave energy spectra.

If a design condition is limited to a certain range of environmental conditions, then these limitations are applied in the evaluation of the environmental loads for that design condition, rather than the actual environmental data for the site or region. Care should be taken to choose the most unfavourable combination of environmental conditions from the specified range.

The description of waves is related to the method chosen for the response analysis, see Chapters 3 and 4.

More details for wave, wind and current conditions are given in Classification Note 30.5.

2.2.2 Regular wave parameters

Deterministic methods are used when the sea state is represented by regular waves defined by the parameters:

— wave height, H
— wave period, T

The reference wave height 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 (worldwide operation) the 100 year wave may be taken as:

$$H_{100} = 32 \text{ m}$$

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

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

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

The combinations of wave height and wave period that are considered should imply a value of steepness that is less than the following limit:

$$S = \begin{cases} \frac{1}{7} & \text{for } T \le 6s \\ \dfrac{1}{7 + \frac{0.93}{H_{100}}(T^2 - 36)} & \text{for } T > 6s \end{cases}$$

When H_{100} = 32 m, then the wave height and period combinations on the steepness limit are given by:

$$H = \begin{cases} 0.22T^2 & \text{for } T \le 6s \\ \dfrac{T^2}{4.5 + 0.02(T^2 - 36)} & \text{for } T > 6s \end{cases}$$

where T is in seconds and H is in metres.

The design wave data are represented by the maximum wave height as well as the maximum wave steepness. The wave lengths are selected which are the most critical to the structure or structure part to be investigated.

2.2.3 Irregular wave parameters

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

- significant wave height, H_s
- average zero-up-crossing period, T_z

The design and analyses for ULS and FLS shall, for worldwide operation, be based the scatter diagrams for the North Atlantic given in Classification Note 30.5 Table 3.3.

An appropriate type of wave spectrum should be used.

A Pierson-Moskowitz wave spectrum representing fully developed seas is applicable when the growth of the waves is not limited by the size of the generation area. Unless the spectrum peak period is close to a major peak in the response transfer function (e.g. a resonance peak) the Pierson-Moskowitz spectrum is assumed to give acceptable results.

The JONSWAP wave spectrum is a peak enhanced Pierson-Moskowitz spectrum and takes into account the imbalance of energy flow in a sea state when the waves are in the process of growing under strong winds; i.e. the seas are not fully developed. This is the case for extreme wave conditions in the North Sea. The JONSWAP wave spectrum is usually applied for ultimate strength analyses of structures operating in harsh environments.

Sea states comprising unidirectional wind waves and swell should be represented by recognised double peaked spectra for example the spectrum proposed by Torsethaugen. If wind waves and swell with different mean directions are critical, then due account of such conditions shall be made.

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.

For design purposes the maximum wave height H_{max} corresponding to the 90% percentile in the extreme value distribution is used:

$$H_{max} = H_s \sqrt{-0.5 \ln\left(1 - p^{\frac{1}{N}}\right)}$$

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

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

$$H_{max} \approx 2.12 H_s$$

The steepness of a specific sea state is defined by:

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

The sea steepness need not be taken greater than the 100 year sea steepness for unrestricted service (worldwide operation), which normally may be taken as:

$$S_s = \begin{cases} \frac{1}{10} & \text{for } T_z \le 6s \\ \frac{1}{15} & \text{for } T_z \ge 12s \\ \text{Linear interpolation for } 6s < T_z < 12s \end{cases}$$

Then the significant wave height and period combination on the steepness limit are given by:

$$H_s = \begin{cases} 0.156T_z^{\ 2} & \text{for } T_z \le 6s \\ 0.206T_z^{\ 2} - 0.0086T_z^{\ 3} & \text{for } 6s < T_z < 12s \\ 0.104T_z^{\ 2} & \text{for } T_z \ge 12s \end{cases}$$

2.2.4 Extreme wave data

The 100 year return period is used as the basis for structural analyses in the Ultimate Limit States (ULS).

In connection with fatigue analysis (FLS) a return period equal to the required fatigue life is used as the basis for wave load analysis. The required fatigue life is normally equal to the specified service life of the unit; minimum 20 years for new-buildings with planned periodic inspection in sheltered waters, see DNV-OS-C103 Sec.5. For modifications etc. of units in service a reduced fatigue life may be accepted, but should normally not be less than 15 years.

In connection with accidental loads or damaged conditions (ALS) a return period not less than one year is taken as the basis for wave load analysis.

If the limiting operating criteria are given as maximum regular or irregular (significant) wave heights only, the 100 year wave height steepness should still be considered.

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 close to a straight line, see Figure 2-1. Such results are obtained for areas with a homogenous wave climate. Other results may be obtained for areas where the climate is characterised by long periods with calm weather interrupted by heavy storms of short duration.

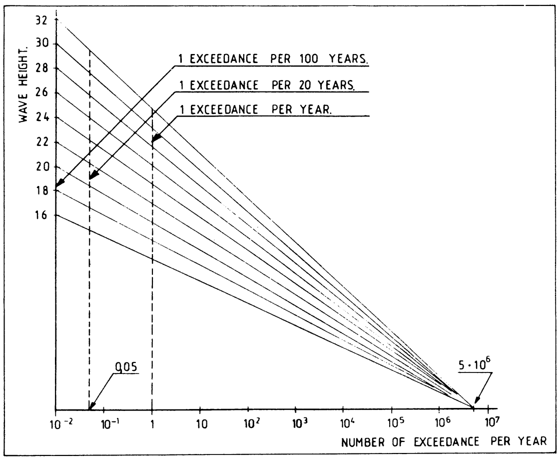


Figure 2-1
Wave height exceedance diagram

2.2.5 Wave theory

When the individual waves have been defined, wave particle motions may be calculated by use of an appropriate wave theory. The linear (Airy) theory is normally sufficiently accurate for column-stabilised units. It may be applied in shallow water, intermediate depth and deep water ranges, whereas other wave theories are often applicable over a more restricted range of depth. Furthermore, the linear approach has been found to be satisfactory for column-stabilised units even when there are quite major departures from the small wave height assumption.

In connection with stochastic response analysis, linear (Airy) theory should always be used.

2.2.6 Long term description of the sea

Long term statistics are associated with non-stationary processes occurring over a period of months and years, whereas short term statistics relate to the stationary processes in periods over only a few hours. In forming a long term statistical description of the sea a suitable statistical model providing a joint probability distribution of wave height and wave period is required.

Long term data for wave conditions are commonly given in the form of a scatter diagram for significant wave height and wave zero-up-crossing period or peak period. The North Atlantic scatter diagram, described in DNV Classification Note 30.5, shall be applied for worldwide operation (see 2.1). Site specific scatter diagrams may be used for restricted operation. FLS response calculations may be directly based on the sea-states represented in the scatter diagram, or a representative condensation into a smaller number of states. ULS calculations should be based on a joint distribution function fitted to the wave data, to take account of the possibility of more unfavourable waves than have been recorded in the observed data set. The three parameter Weibull distribution is commonly used to describe the marginal distribution of significant wave heights, and a conditional log-normal distribution is often used to describe the wave periods.

The FLS response analysis should cover the range of probability levels from 10^{-1} to 10^{-4}, for exceedance of stress ranges in the platform lifetime.

The ULS response analysis should cover the range from 10^{-1} to 10^{-8}, for exceedance of load effect maxima in the platform lifetime.

2.2.7 Wave energy spreading function

In stochastic wave load analysis the effect of wave short-crestedness may easily be included by introducing a wave energy spreading function. The Pierson-Moskowitz wave spectrum together with a $\cos^4\alpha$ wave spreading function should be utilised in the fatigue analyses (FLS) of column-stabilised units, see DNV-OS-C103 Sec.5. α is angle between direction of elementary wave trains and the main direction of the short-crested wave system.

For the extreme wave analysis (ULS) the wave spreading function should be according to the designer's specification or $\cos^{10}\alpha$, see Classification Note 30.5 3.2.

2.3 Wave loads

2.3.1 Worldwide operation

The wave loads acting on column-stabilised units intended for operation worldwide without any restriction, should be analysed by use of a diffraction model, which takes into account the reflection of waves. For preliminary design, Morison's equation may be applied, together with a contingency factor, see 2.3.3.

2.3.2 Site specific harsh environment

The wave loads acting on column-stabilised units intended for operation in a site specific harsh environment area, should be analysed by use of a diffraction model. For preliminary design, a simplified Morison's equation may be applied, together with a contingency factor, see 2.3.3.

2.3.3 Benign waters

The wave loads acting on twin pontoon column-stabilised units intended for operation in benign waters and following normal class survey intervals, may be calculated with a simplified model based on Morison's equation.

When a Morison model is utilised, a contingency factor of 1.3 for ULS and 1.1 for FLS shall be applied, see also DNV-OS-C103 Sec.4 B107, DNV-OS-C103 Sec.5 B403 and DNV-OS-C103 Appendix B.

Design for benign waters shall be based on specific environmental data for the area the unit shall operate. Due consideration shall be given to environmental loads caused by swell and currents.

2.3.4 Morison equation

The Morison equation is given in Classification Note 30.5 6.1, including inertia and drag forces. This equation is valid for slender structural elements, for example wave length/diameter ratio above 5.

It is important when calculating added mass for the pontoon that interaction effects between column and pontoon, end effects, and the effect of rounded cross sectional corners are taken into account.

2.3.5 Linearisation of the drag force

Non-linear, hydrodynamic drag forces acting on braces should be accounted for. The drag force on submerged braces may be linearised by means of recognised methods. It is normally acceptable to linearise the drag force to account for the correct force under the maximum water particle speed due to the waves, calculated in accordance with linear wave theory, or conservatively taken as 5 m/s. The wave loads acting on the pontoons are normally dominated by hydrodynamic inertia forces, so the linearisation of the drag term for the pontoons is less critical.

It is advisable to check that the modelling of drag loads provides a damping effect that leads to realistic heave response at resonance.

2.3.6 Asymmetry factor in air gap calculation

Generally a wave asymmetry factor of 1.2 should be applied in the air gap calculations unless model tests are available. In this case the air gap shall be calibrated against the model tests.

Calculations for sufficient air gap is further referred to in 6.3.

3. Design Loads

3.1 Introduction

As described in DNV-OS-C101 and DNV-OS-C103, the following load categories are relevant for column-stabilised 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.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.

3.2 Loads to be applied in global and local models

Analytical models shall adequately describe the relevant properties of loads, load effects, stiffness, and displacement, and shall satisfactorily account for the local and system effects of time dependency, damping, and inertia.

It is normally not practical, in design analysis of column-stabilised units, to include all relevant loads (both global and local) in a single model. Generally, a single model would not contain sufficient detail to establish local responses to the required accuracy, or to include consideration of all relevant loads and combinations of loads. Assessment of single model solutions is further discussed in 4.8.2.

It is often more practical, and efficient, to analyse different load effects utilising a number of appropriate models and superimpose the responses from one model (global) with the responses from another model (local) in order to assess the total utilisation of the structure.

The modelling guidance given in 3.8.6, Chapter 4, and Chapter 5 can be considered as a proposed use of different models in accordance with an acceptable analytical procedure. The procedures described are not intended to restrict a designer to a designated methodology when an alternative methodology provides for an acceptable degree of accuracy, and includes all relevant load effects. Further, the modelling procedures and guidance provided are intended for establishing responses to an acceptable level of accuracy for final design purposes.

For preliminary design, simplified models may be used in order to more efficiently establish the design responses, and to achieve a simple overview of how the structure responds to the design loads.

3.3 Permanent loads (G)

Permanent loads are described/defined in DNV-OS-C101 Sec.3 C and DNV-OS-C103 Sec.3 C.

Hydrostatic sea pressure for local analyses and scantlings of tanks are given in 3.8.

3.4 Variable functional loads (Q)

Variable functional loads are described/defined in DNV-OS-C101 Sec.3 D and DNV-OS-C103 Sec.3 D.

Variations in operational mass distributions, especially in the pontoons with maximum and minimum ballast, shall be adequately accounted for as part of the global load effects for the structural design.

Tank pressures for local analyses and scantlings are given in 3.8.

3.5 Environmental loads (E)

3.5.1 General

Environmental loads are in general terms given in DNV-OS-C101 Sec.3 E and DNV-OS-C101 Sec.3 F and in DNV-OS-C103 Sec.3 E.

Wave loads are given in Chapter 2.

Practical information regarding environmental loads is given in the Classification Note 30.5.

Hydrodynamic sea pressure and vertical accelerations form an integrated part of tank pressures and sea pressures for local analyses and scantlings of tanks as given in 3.8.

3.5.2 Current loads

Current loads may normally be calculated from the drag term in the Morison equation. The variation in current profile with water depth may be determined in accordance with Classification Note 30.5.

The global response from the current loads on a column-stabilised unit is negligible compared with the response from wave loads for consideration of overall hull strength. The main contribution of current is to the reaction forces of anchor lines and thrusters, see 3.5.5. Current loads need therefore normally not be considered for global and local structural analyses of column-stabilised units.

3.5.3 Wind loads

The horizontal wind force on a column-stabilised unit for a maximum sustained wind speed will cause the unit to heel, with a maximum heel angle of the unit of order 10-15º. Such heel angle would give a considerable sideway gravitational deck load component.

In practice, however, sustained heel of the platform due to wind is kept less than about 3º by re-ballasting and by anchor forces, and it is considered too conservative to add the maximum theoretical wind heeling effect to the 100 year wave forces. Hence, in the global analysis of the platform wind forces may normally be neglected.

3.5.4 Pressure height for sea pressure

Sea pressure (static and dynamic) shall include the effect of wave height and relative motion of the platform.

To be consistent with the analysis of global wave forces, the effect of reduced wave particle motion with increasing depth may be included (Smith effect).

Due to the requirement for positive air gap, see DNV-OS-C103 Sec.4 D100, the pressure height may be calculated as:

$$h = h_0\left(1 - \frac{a}{h_0}\left(1 - e^{-k(h_0 - a)}\right)\right) = C_w h_0$$

h_0 = distance from load point to underside of lowest deck
a = wave amplitude

$k = \frac{2\pi}{\lambda}$

λ = wave length

C_w = reduction factor due to wave particle motion (Smith effect).

A value of $C_w = 0.9$ may be used for any wave steepness or distance to load point unless otherwise documented.

3.5.5 Reaction forces of anchor lines and thrusters

For global structural analysis of slack moored column-stabilised units (ring pontoon and twin pontoon) the reaction forces of the anchor lines and thrusters are of minor importance.

The slack moored anchor line contribution to axial forces in the bracing system (for units with twin pontoon configuration) is not more than about 10% of the wave forces. Hence, by taking into account the phase lag between the reaction forces and wave forces the increase in the total wave forces will be very small and may normally be neglected in the global analysis.

For global structural analysis of taut moored column-stabilised units the reaction forces of anchor lines may be of importance, as it will affect the motion of the unit. Hence the mooring stiffness, pretension and possibly downset shall be included in the analysis, unless it can be documented that the effect is insignificant.

For local strength analysis of columns in way of fairlead and windlass the design should be based upon the breaking strength of anchor lines, see 6.1.

3.6 Accidental loads (A)

Accidental loads are in general terms given in DNV-OS-C101 Sec.3 G, and in DNV-OS-C103 Sec.3 G.

Recommendations for generic accidental loads are given in DNV-OS-A101 Sec.2 G.

Requirements for the Accidental Limit State events for structural design of column-stabilised unit are given in DNV-OS-C103 Sec.6. Requirements for structural redundancy for typical twin pontoon units are given in DNV-OS-C103 Sec.7.

Recommendations for the tank and sea pressures for the accidental heeled condition are given in 3.8.5 and 3.8.6.

Guidance for the assessment of structural redundancy and heel after loss of buoyancy is given in 5.2.7.

3.7 Deformation loads (D)

Deformation loads are specified in DNV-OS-C101 Sec.3 H, and in DNV-OS-C103 Sec.3 F.

When relevant, depending on the procedure for deck mating to hull, the effects of built-in stresses due to mating shall be accounted for, for example by separate global analyses.

Other relevant deformation load effects may include those resulting from temperature gradients, for example when hot-oil is stored in a compartment adjacent to the sea.

3.8 Tank pressures and sea pressures

3.8.1 General

For the design and scantlings of tanks, both ballast tanks and other tanks in hull and deck, local tank loads are specified as tank pressure and sea pressure in 3.8.2 to 3.8.5. Typical combinations for local tank and sea pressures are given in 3.8.6.

Note that the design and scantlings shall include the effects of relevant global and local responses, see 5.3.

The tank testing conditions should as a minimum, represent the maximum static pressure during operation. Requirements to testing for watertightness and structural tests are given in DNV-OS-C401 Ch.2 Sec.4. For arrangements with free flooding or level alarms installed, limiting the operational tank pressures, it shall be ensured that the tank will not be over-pressurised during operation and tank testing conditions.

Tanks are to be designed for the maximum filling height. The following tank filling types are defined:

Alternative 1: For tanks with maximum filling height to the top of the air pipe:

— Applicable for arrangements with no limitations of the possible filling height.
— The tank is filled by pumps.
— In addition to the static pressure head to the top of the air-pipe, the dynamic pressure head due to flow through air pipes due to the operation of the pumps (P_{DYN}) should be considered.

Alternative 2: For tanks with maximum filling height less than to the top of the air pipe:

a) filling with pumps with tank level alarms installed:

— Applicable for arrangements with limitations of the possible filling height.
— The tank is arranged with an alarm system installed to limit the maximum pressure height.
— Criteria applicable for the tank filling arrangements are given in DNV-OS-D101 Ch.2 Sec.3 C300.
  Such arrangement should have a high level alarm, and a high-high level alarm with automatic shut-off of the pump. Consequence of possible failure of the alarm system may be considered as an accidental event.
— The dynamic pressure head due to the operation of the pumps (P_{DYN}) may normally be neglected, provided the shut-off level is set to 98 % of the tank height.

b) filling by free flooding:

— Applicable for arrangements where the tanks are filled by gravity, without pumps.
— Criteria applicable for the tank filling arrangements are given in DNV-OS-D101 Ch.2 Sec.3 C300, for free flooding ballast systems.
— The dynamic pressure head due to the operation of the pumps (P_{DYN}) may be neglected.

3.8.2 Tank pressures, ULS

Tank loads for local analyses and scantlings are given in DNV-OS-C103 Sec.3 D300 and in the following.

For design of all tanks the following internal design pressure condition shall be considered:

$$p_d = \rho g_0 h_{op} \left(\gamma_{f,G,Q} + \frac{a_v}{g_0}\gamma_{f,E}\right) \quad (kN/m^2)$$

a_v = maximum vertical acceleration (m/s^2), being the coupled motion response applicable to the tank in question. For preliminary design calculations of tank pressures, a_v may be taken as $0.25g_0$. For final design, a_v shall be documented

g_0 = 9.81m/s^2 , acceleration due to gravity

ρ = density of liquid, minimum density equal to that of seawater (1.025 kg/m^3)

$\gamma_{f,G,Q}$ = load factor for permanent and variable functional loads, see DNV-OS-C103 Sec.4 Table A1 for ULS

$\gamma_{f,E}$ = load factor for environmental loads, see DNV-OS-C103 Sec.4 Table A1 for ULS

h_{op} = vertical distance (m) from the load point to the position of maximum filling height. For tanks adjacent to the sea that are situated below the extreme operational draught (T_E), the maximum filling height for ULS design is not to be taken less than to the extreme operational draught, see DNV-OS-C101 Sec.3 D311 and section 3.8.1.

For tanks where the air pipe may be filled during filling operations, the following additional internal design pressure conditions shall be considered:

$$p_d = (\rho g_0 h_{op} + p_{dyn})\gamma_{f,G,Q} \quad (kN/m^2)$$

p_{dyn} = pressure (kN/m^2) due to flow through pipes, minimum 25 kN/m^2

This internal pressure need not to be combined with extreme environmental loads, because they it is not likely that the tank filling operation will occur together with extreme waves. Normally only static global response need to be considered.

Parameters for tank pressures are illustrated in Figure 3-1. Refer also to tank pressures in damaged compartments specified in the ALS heeled condition, see 3.8.5.

The load points for plate fields, for stiffeners, and for girders are defined in DNV-OS-C103 Sec.3 B100.

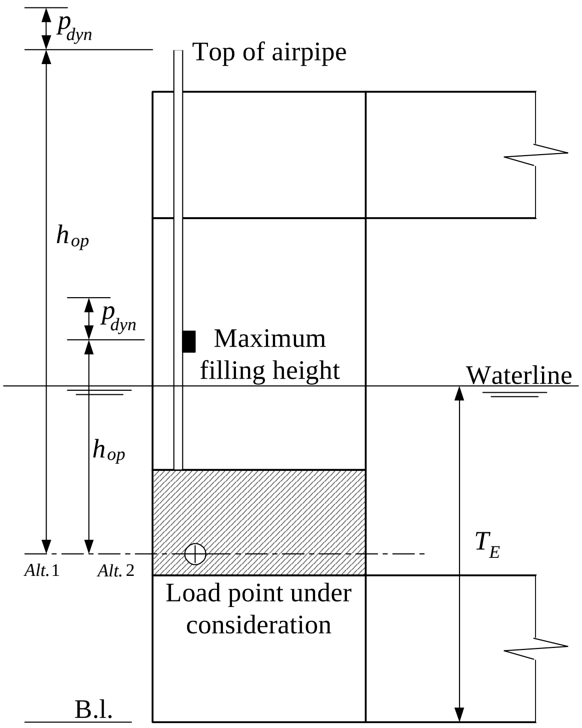


Figure 3-1
Parameters for tank pressures

3.8.3 Sea pressures, ULS

Sea pressures for local analyses and scantlings are given in DNV-OS-C103 Sec.3 E200 and in the following.

The design sea pressure acting on pontoons and columns of column-stabilised units in operating conditions shall be taken as:

$$p_{d,ULS} = p_s\gamma_{f,G,Q} + p_e\gamma_{f,E} \quad (kN/m^2)$$

p_s = $\rho g_0 C_w (T_E - z_b)$ (kN/m^2) ≥ 0
p_e = $\rho g_0 C_w (D_D - z_b)$ (kN/m^2) for $z_b \geq T_E$
p_e = $\rho g_0 C_w (D_D - T_E)$ (kN/m^2) for $z_b < T_E$
T_E = extreme operational draught (m) measured vertically from the moulded baseline (B.l.) to the assigned load waterline
C_w = reduction factor due to wave particle motion (Smith effect, see 3.5.4) C_w = 0.9 unless otherwise documented
D_D = vertical distance (m) from the moulded baseline to the underside of the deck structure. (The largest relative distance from the moulded baseline to the wave crest may replace D_D if this is proved smaller.)
z_b = vertical distance (m) from the moulded base line to the load point
p_s = static sea pressure
p_e = dynamic (environmental) sea pressure.

The load factors are given in DNV-OS-C103 Sec.4 Table A1.

Parameters for sea pressures are illustrated in Figure 3-2.

The Smith effect (C_w = 0.9) shall only be applied for loading conditions including extreme wave conditions, i.e. ULS a) and b) loading conditions.

Relevant combinations of tank and sea pressures, with combinations of maximum/minimum pressures, are specified and discussed in 3.8.6 and 5.3.

3.8.4 Sea pressure at wave trough, ULS

The equations for p_e and p_s in 3.8.3 correspond to the maximum sea pressure (wave crest elevation).

In combination with the maximum tank pressures, the external sea pressure up to the lowest waterline at wave trough may be applied in the design for the external plate field boundaries.

Such external sea pressure may be taken up to half the pontoon height, see Figure 3-2. The design sea pressure up to the lowest waterline is regarded as permanent load and may be taken as:

$$p_d = \rho g_0\left(\frac{h_p}{2} - z_b\right)\gamma_{f,G,Q} \quad (kN/m^2) \quad \text{for} \quad z_b < h_p/2$$

h_p = height of pontoon (m)

The load factor is given in DNV-OS-C103 Sec.4 Table A1.

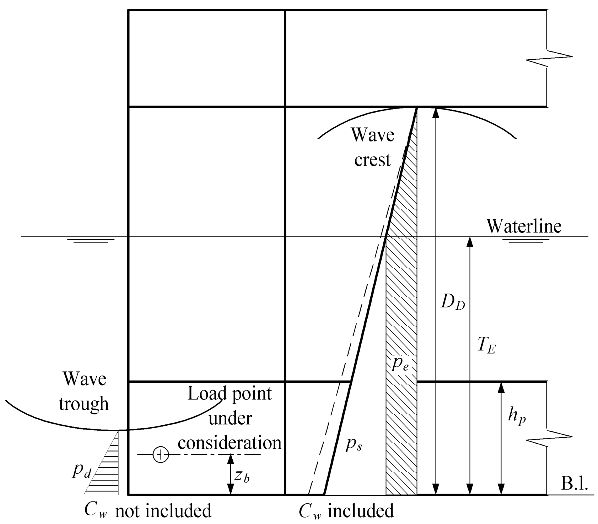


Figure 3-2
Parameters for sea pressures

3.8.5 Sea pressures, ALS heeled condition

For ALS design with 17° heeled condition as given in DNV-OS-C103 Sec.6 F, the design sea pressure can be expressed as:

$$p_{d,\,ALS} = \rho g_0 h_{17} \gamma_{f,\,A} \quad (\mathrm{kN/m^2})$$

h_{17} = vertical distance (m) from the load point to the damaged heeled condition still water line after accidental flooding (maximum heel 17°, effect of submersion included, see also 5.2.7 item 2 and DNV-OS-C301 Ch.2 Sec.1 E400).

$\gamma_{f,A}$ = load factor in the ALS is 1.0, see DNV-OS-C101 Sec.2 D700.

The sea pressure in heeled condition is illustrated in Figure 3-3. For ALS design, the Smith effect C_w = 1.0. Note that the sea pressure for ALS heeled condition shall also be applied as internal tank pressure on bulkheads and decks surrounding the damaged compartment(s).

Heeling of the unit after damage flooding shall be accounted for in the assessment of structural strength. The unit shall be designed for environmental condition corresponding to 1 year return period after damage. To simplify the design approach, the environmental loads may be disregarded if the material factor is taken as γ_M =1.33, see DNV-OS-C103 Sec.6 F.

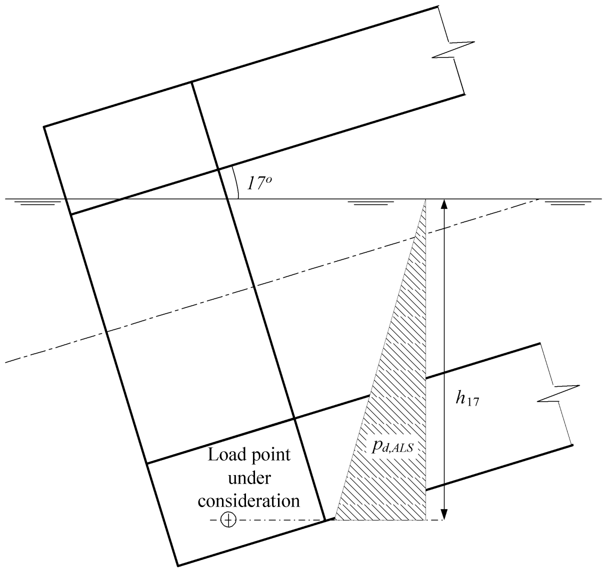


Figure 3-3
Sea pressures in heeled condition

3.8.6 Combination of tank and sea pressures

Local structural models should be created in order to evaluate responses of the structure to various sea and tank pressures. Examples of considerations that should be evaluated in connection with the load cases of local pressures acting on pontoon and column sections of a column-stabilised unit are given below:

— The intention of the local model is to simulate the local structural response for the most unfavourable combination of relevant local loads. Relevant combinations of internal (tank) and external (sea) pressures for tanks should be considered for both the intact and damage load conditions.
— If cross-section arrangements change along the length of the structure, several local models may be required in order to fully evaluate local response at all relevant sections.
— For tanks separated by internal watertight bulkhead/deck, the internal tank pressure should normally not be considered to act simultaneously on both sides of the bulkhead/deck. Combinations with maximum tank pressure from each of the tanks and zero tank pressure from the neighbouring tank should be considered. Effects of sea pressure response on the internal watertight bulkhead/deck should be assessed and included when relevant.
— For some structural elements, for example access/pipe tunnel in the pontoon and some girder configurations, the effect of simultaneous tank pressure in neighbouring tanks may be governing for the design. Effects of simultaneous sea pressure response should be assessed and included when relevant.
— For external structural components (stiffened plates adjacent to sea), the maximum external (sea) pressure and the maximum internal (tank) pressure will normally not act simultaneously.
— Loads are usually applied in the analysis models at the girder level and not at the individual stiffener level (typical global and in some cases local analysis models). In such cases the local stiffener bending is not included in the model responses. The stiffener bending response will then be explicitly included in the buckling code check as lateral pressure (for plate induced and/or stiffener induced buckling).
— For transversely stiffened structures (i.e. girders orientated in the transverse direction) the local responses extracted from the local model are normally responses in the structural transverse direction (see σ_y in Figure 5-5). Shear and bending responses in girders shall also be considered.
— For structural arrangements with continuous, longitudinal girder arrangements, a longitudinal response will also be of interest (see σ_x in Figure 5-5).
— For structural transverse sections without continuous longitudinal girder elements, two-dimensional structural models may be considered as being adequate.
— For space frame beam models, relevant consideration shall be given to shear lag effects.

Recommended design pressure combinations for local tank and sea pressures are given in Table 3-1.

Table 3-1 Design pressure combinations for tank pressure and sea pressure

Limit state	*External or internal structural component*	*Desgn pressure combination*
Tank testing		Refer DNV-OS-C401 Ch.2 Sec.4, Testing of Watertightness and Structural Tests. Note DNV-OS-C103 Sec.3 D307: *"In cases where the maximum filling height is less than the height to the top of the air pipe, it shall be ensured that the tank will not be over-pressured during operation and tank testing conditions".*
ULS both a) and b)	Internal	I_{max}: p_d and E = 0 [1)] E_{max}: $p_{d,ULS}$ and I = 0 [2)]
	External	I_{max}: p_d and E = 0 [3)] E_{max}: $p_{d,ULS}$ and I = 0
ALS Heeled 17° (γ_M = 1.33)	Internal	I_{max}: $p_{d,ALS}$ (for damaged compartments) and E = 0 [1)]
	External	E_{max}: $p_{d,ALS}$ and I = 0

Definitions:
External structural component is (stiffened) plate panel adjacent to sea.

I_{max} = maximum internal pressure (tank pressure); see 3.8.2 for ULS and 3.8.5 for ALS damaged compartment.

E = the simultaneous and corresponding external pressures.

E_{max} = maximum external pressure (sea pressure), see 3.8.3 for ULS and 3.8.5 for ALS.

I = the simultaneous and corresponding internal pressures.

1) For the considered internal structures, effect of relevant simultaneous/corresponding *external* pressures to be assessed and included.

2) For the considered internal structures, effect of relevant simultaneous/corresponding *internal* pressures to be assessed and included.

3) For the considered external plate field boundaries, sea pressure at wave trough may be applied (see 3.8.4) when considering the maximum tank pressure.

4. Global Response Analysis

4.1 Introduction

Methods and models for global response analyses are outlined in DNV-OS-C103 Sec.4, DNV-OS-C103 Sec.5 and DNV-OS-C103 Appendix B. An appropriate method may be:

— stochastic analysis
— "design wave" analysis
— regular wave analysis.

The characteristic hydrodynamic responses with corresponding input parameters as wave length, height and direction are discussed in 4.6. It should be noted that the maximum global characteristic response might occur for environmental conditions that are not associated with the characteristic, largest, wave height. In such cases, wave period and associated wave steepness parameters are governing factors for the maximum and minimum responses.

Stochastic methods for fatigue analysis (FLS) are recognised as the best methods for simulating the irregular nature of wave loads. Motion characteristics are determined by stochastic methods by using relevant site specific data or North Atlantic environmental data for worldwide operation. Simplified fatigue analyses should be used as screening process to identify locations for which a detailed stochastic fatigue analysis should be undertaken.

For structural design evaluation, engineering judgement and knowledge of structural behaviour is vital for designing a sound and safe unit. For this purpose, stochastic stress results are not well suited, as simultaneity of force and stress distribution is lost, making it difficult to judge the most effective ways of improving the structure. Application of "design wave " approach or regular wave analyses are effective methods for design evaluation and engineering judgement.

4.2 Stochastic analysis

Stochastic analysis applies the statistical distributions of the waves for calculation of short term and long term responses.

A *frequency domain procedure* is the most suitable for response analysis of column-stabilised units and is described below:

a) Calculation of motion, hydrodynamic load and acceleration in regular waves for several wave lengths and wave headings - establishing transfer function.

b) Transfer functions for stresses at a number of specified points to be calculated based upon loads and accelerations calculated under a), see Figure 4-1 (1-2).

c) By combination with different wave spectra (1) the response spectra to be established (for motion, loads or stresses) for different headings, see Figure 4-1 (3).

d) By integrating these response spectra, the short term response parameter (Raleigh parameter R) and significant response, σ_s, may be calculated:

$$R = \sqrt{2 \cdot \mathrm{Area}} \quad \text{(Raleigh parameter)}$$

By repeating this procedure for various wave parameters (H_s, T_z) the energy operator may be established, see Figure 4-1 (4). β = heading angle:

$$\sigma_s / H_s = f(T_z, \beta)$$

e) The maximum response corresponding to the 90% fractile in the extreme value distribution (short term response):

$$\mathrm{Resp(max)} = R\sqrt{\ln N} = \sigma_s \sqrt{-0.5 \ln\left(1 - p^{\frac{1}{N}}\right)} = 2.12\,\sigma_s$$

N = number of waves (1080 in a 3 hour storm)
p = fractile level, (1 – p) = probability of exceedance

f) By taking into account the probability for different wave headings, the maximum response amplitude, 90% percentile, for a specified sea state may be derived.

g) Further, by taking into account the probability of different wave spectra, the long term response distribution of the response in question to be established, see Figure 4-1 (5-6). The slope of this long term distribution is an important parameter for the fatigue analysis, see Figure 4-1 (6).

Scatter diagram for the North Atlantic, applicable for worldwide operation, is given in Classification Note 30.5 Table 3.3.

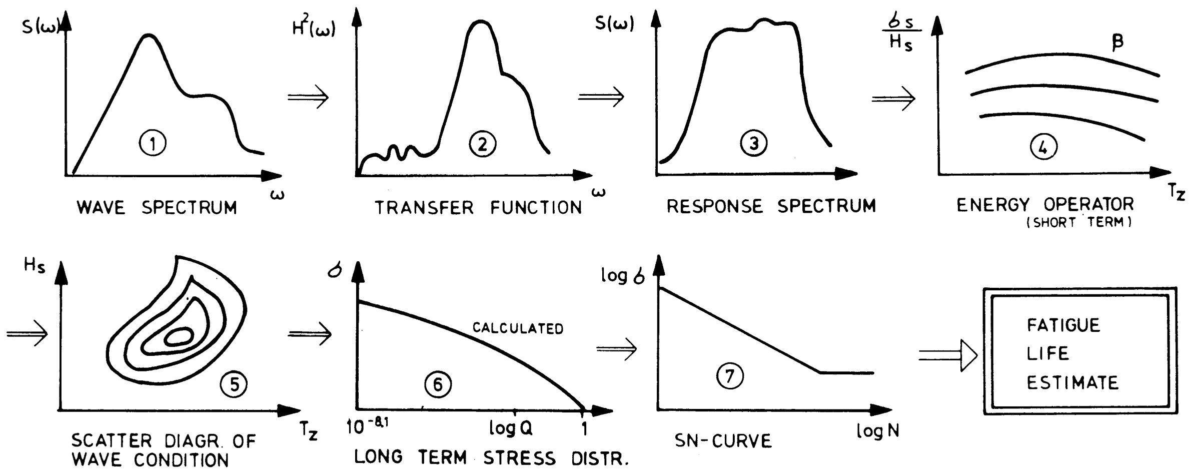


Figure 4-1
Stochastic stress analysis procedure

4.3 Design wave analysis

To satisfy the need for simultaneity of the responses, a design wave approach is often adopted for maximum stress analysis of column-stabilised units. The merits of the stochastic approach are retained by using the extreme stochastic values of some characteristic response parameters in the selection of *design wave parameters.*

The method for evaluation of the design wave parameters will be:

a) Characteristic response parameters and corresponding *wave headings* may be chosen according to experience with stochastic wave analysis, see 4.6. Typical characteristic response parameters are shown in Figure 4-3.

b) The maximum response parameter (*Resp (max)*), 90% percentile, is calculated by using the stochastic short term response analysis, see 4.2 and Figure 4-2.

c) The *wave length*, λ_d , of the design wave may be evaluated from the transfer function of the response parameter in question. Normally the wave length should correspond to the peak-wave-length or slightly higher, see Figure 4-2 (2).

d) The *wave amplitude*, a_d , may be calculated by using the most probable largest response amplitude and the value of the transfer function corresponding to the selected wave length.

The wave amplitude may be calculated according to the following formula:

$$a_d = \frac{\mathrm{Resp(max)}}{\mathrm{TR}}$$

a_d = Wave amplitude
TR = Response (from unit wave amplitude) for relevant wave length λ_d

Guidance note:

If the regular wave steepness (given in 2.2.2) exceeds the extreme 100 year steepness, then the selected λ_d is not the most critical for the characteristic response parameter.

---e-n-d---of---G-u-i-d-a-n-c-e---n-o-t-e---

e) The design waves are used in the calculation of hydrodynamic forces for further input to the global structural model, see sections 4.6 and 4.8.

f) Each loadcase corresponding to one wavelength and one heading should be calculated at two time instances, called a real and an imaginary part. The real part may correspond to a wave crest amidships and the imaginary part to a wave zero-crossing at the same point. Linear load effect (E) at any time instance in the wave (with frequency ω) can be obtained by combination of real (R) and imaginary (I) load effects:

$$E(t) = R\cos(\omega t) + I\sin(\omega t) = E_{max}\cos(\omega t + \varphi)$$

and

$$E_{max} = \sqrt{R^2 + I^2}$$

φ = arctg $^{I}/_{R}$ (phase lag)

g) As an alternative to calculate the maximum response based on a *short term distribution* one may apply a *long term distribution* with applicable scatter diagrams.

The following partial factors shall be applied:

— When Morison model is utilised, a contingency factor of 1.2 shall be applied, see DNV-OS-C103 Appendix B, in addition to the relevant load factors specified in DNV-OS-C103 Sec.4 A.

— The Weibull distribution parameter h for simplified fatigue analysis should have a value h = 1.1 in combination with worldwide operation criteria. Alternatively the Weibull distribution parameter h may be calculated based on site specific criteria.

See DNV-OS-C103 Sec.5 and DNV-OS-C103 Appendix B.

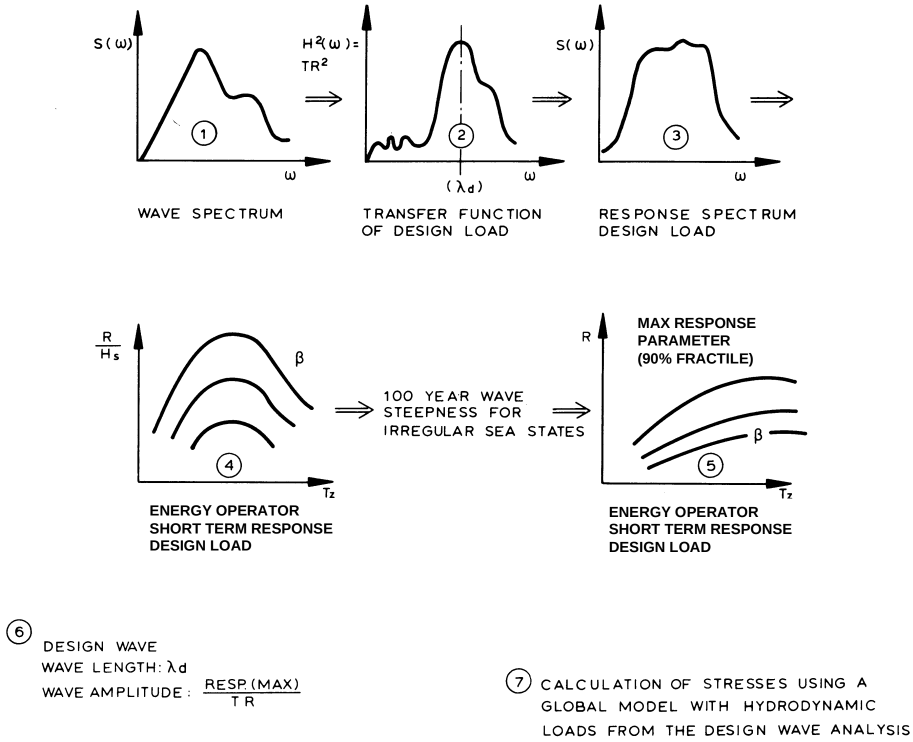


Figure 4-2
Design wave analysis procedure

4.4 Simplified design wave analysis

This method is very similar to the design wave analysis. The method for establishing the design wave, however, is modified.

The characteristic response parameters and corresponding wave headings and wave lengths, λ_d , may be chosen according to experience with the stochastic- and design wave analysis.

The wave amplitude, a_d , may be calculated according to formulae for maximum hundred year steepness for regular waves or calculated based on experience from using the design wave approach.

This method may be applied for benign waters.

4.5 Methods and models

Guidance for the selection of methods and models to be applied in the design of typical column-stabilised units are given in DNV-OS-C103 Appendix B.

The following methods and models are identified:

— Hydrodynamic models:

1) Hybrid model - Sink-source and/or Morison (when relevant, for calculation of drag forces) (*)
2) Morison model

— Global structural strength models:

3) Beam model
4) Combined beam and shell model. The extent of the beam and shell models may vary depending on the design. For typical beam structures a beam model alone may be acceptable
5) Complete shell model (*)

— Fatigue method:

6) Simplified fatigue analysis
7) Stochastic fatigue analysis, based on screening process with simplified approach to identify critical details (*).

Methods and models with (*) should be applied for ring pontoon units. The selection depends on the following parameters:

— Type of unit: twin pontoon or ring pontoon
— Member size of structure (slender or full-bodied)
— Environmental condition: worldwide operation, site specific harsh environment or benign waters, restricted area
— Inspection and maintenance survey: normal class survey intervals (sheltered water/dry-dock), or survey at location (permanently installed units).

4.6 Characteristic global hydrodynamic responses

4.6.1 General

The following responses will normally be governing for the global strength of the unit. The responses are illustrated on a typical twin pontoon unit with horizontal braces between the pontoons, and diagonal braces between deck and pontoon, see Figure 4-3.

Similar responses are also applicable for the design of ring pontoon units, applying split force (F_S), torsion moment (M_t) and shear force (F_L) related to both transverse and longitudinal axes. This to account for responses in both transverse and longitudinal pontoons of a ring pontoon unit.

1) Split force between pontoons, F_S
2) Torsion moment about a transverse horizontal axis, M_t
3) Longitudinal shear force between the pontoons, F_L
4) Longitudinal acceleration of deck mass, a_L
5) Transverse acceleration of deck mass, a_T
6) Vertical acceleration of deck mass, a_V.

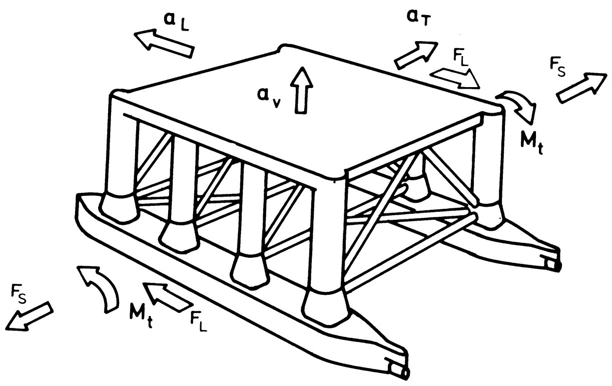


Figure 4-3
Characteristic hydrodynamic responses

The forces F_S and F_L and the torsion moment M_t may be calculated by integrating the forces acting on all members located on one side of the centre plane. The responses are normally calculated with respect to a point located on the centreline at still water plane and above the centre of gravity.

The accelerations may be calculated for a point located at the centre of the deck area. The longitudinal and transverse acceleration of the deck should include the weight component due to pitch and roll respectively.

Particular attention shall be given to the combined response of split and longitudinal shear force. Generally, it will be found that maximum longitudinal shear forces occur on a different wave heading than that heading providing maximum response when simultaneous split forces are taken into account.

In addition these responses may be used to establish design wave data and limiting environmental criteria for transit condition, see 6.4.

4.6.2 Split force between pontoons, F_S

The critical value for this response will occur at a wave heading θ = 90° (beam sea) and a wave length of approximately twice the outer breadth between the pontoons. Consequently the wave height may be found from the steepness relations given in 2.2, or the critical responses may be found by using the stochastic approach as described in 4.3, see Figure 4-4.

A typical example of a transfer function for this response is shown in Figure 4-5.

This response will normally give the maximum axial force in the transverse horizontal braces of a twin pontoon unit. For a unit without these braces, this response will give maximum bending moment for the transverse deck structure. In addition to the axial force, which is a global response, local vertical drag and inertia force on the bracing structure should be accounted for.

For a ring pontoon unit, this response will give axial force and bending moment in the pontoons (about pontoon vertical and transverse axes), with maximum responses both at pontoon mid-section and at pontoon end.

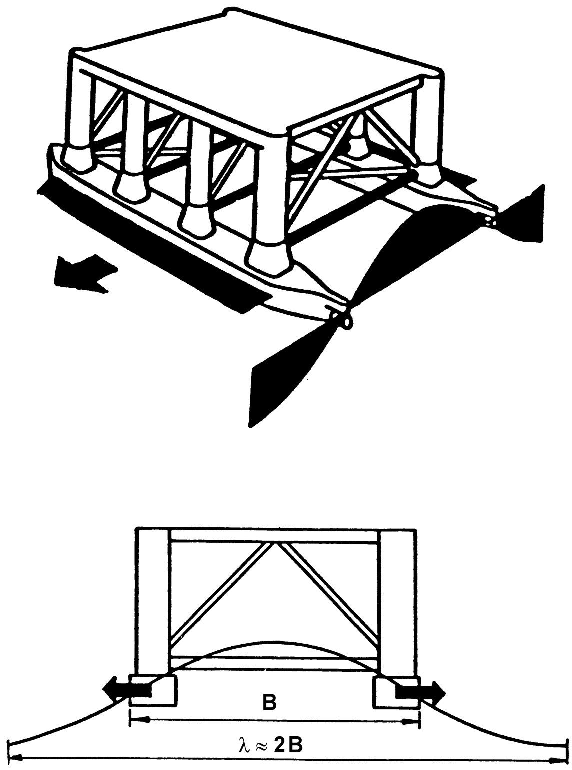


Figure 4-4
Split force between pontoons

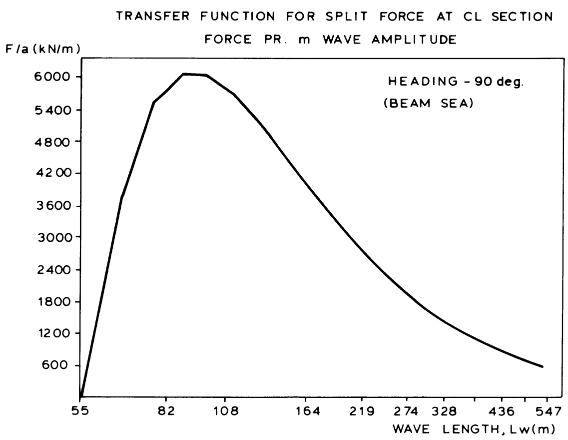


Figure 4-5
Transfer function for split forces (example)

4.6.3 Torsion moment, M_t

The critical value for this response will normally occur at a wave heading between θ = 45° to θ = 60° (diagonal sea) and a wave length of approximately the distance of the diagonal between the pontoon ends. The wave height may be found from the steepness relation given in 2.2, or the critical responses may be found by using the stochastic approach as described in 4.3, see Figure 4-6 and Figure 4-7.

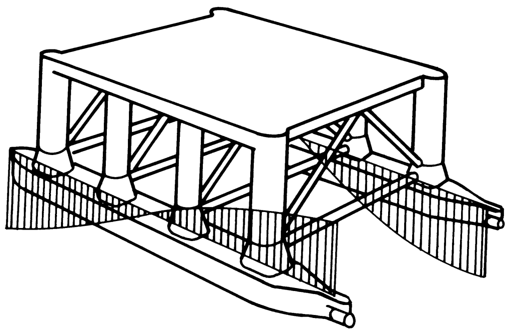
A typical example of a transfer function for this response is shown in Figure 4-8.

This response will normally give the maximum axial force in the diagonal horizontal and diagonal vertical braces of a conventional twin pontoon unit. For units without these braces, the main deck structure has to be designed for this moment.

In addition to the torsion moment, response from simultaneous split force should be accounted for. The bending moment is derived from the torsion moment, which is not sensitive to the choice of accurate wave heading. The transverse force is derived from the simultaneous split force which is sensitive for the choice of wave heading, (e.g. in a typical case 50% increase when for example θ varies from 45° to 55°). The bending moment and transverse force will give added stresses, and therefore it is important to select the correct θ, which theoretically has to be somewhat above the heading giving maximum torsion moment. It may therefore be necessary to evaluate more than one heading.

In addition to the global response, local drag and inertia force on the bracing structure (twin pontoon units) should be accounted for when relevant.

For a ring pontoon unit, the torsion moment will give maximum responses at the pontoon/node/column intersections and at the column to deck connections. For a typical ring pontoon unit, the torsion moment will give less responses than split forces and longitudinal/transversal shear forces.



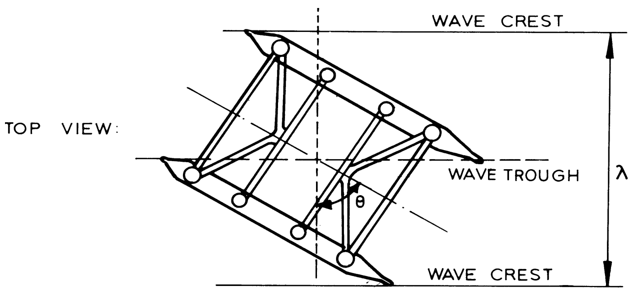


Figure 4-6
Torsion moment

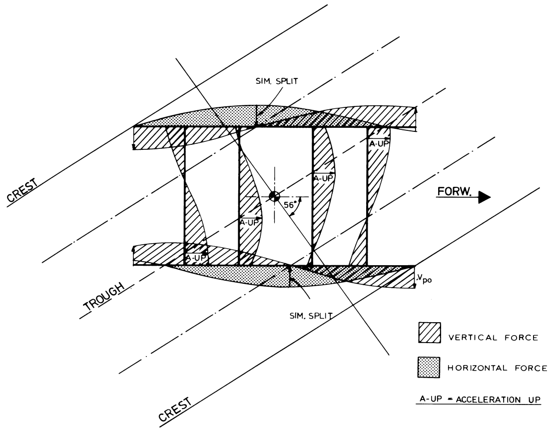


Figure 4-7
Platform in diagonal wave

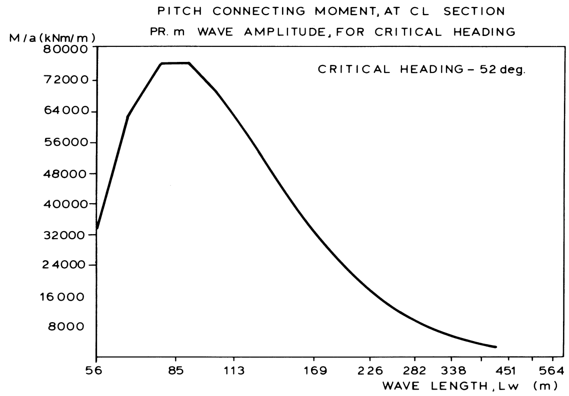


Figure 4-8
Transfer function for torsion moment (example)

4.6.4 Longitudinal shear force between the pontoons, F_L

The critical value for this response will normally occur at a wave heading between θ = 45° to θ = 60° (diagonal sea), and the wave length is about 1.5 times the distance of the diagonal between pontoon ends. The wave height may be found from the steepness relation given in 2.2, or the critical response may be calculated using the stochastic approach as described in 4.3, see Figure 4-9.

This loadcase contains the same force components as for the torsion moment case, but the longitudinal forces on pontoons and columns are maximised. In this case the response will introduce opposite longitudinal (and vertical) displacement for each pontoon, and thus introduce bending moment (S-moment) on the transverse braces, see Figure 4-10.

For typical twin pontoon units with bracing structures, this loadcase is normally the governing loadcase for all horizontal bracing structures. In this case it is important to select the correct critical wave direction. Also in this case, responses from simultaneous split force should be accounted for.

In addition to the longitudinal shear force, response from simultaneous split force should be accounted for. The transverse force is derived from the simultaneous split force which is sensitive to the choice of wave heading, (e.g. in a typical case 50% increase when for example θ varies from 45° to 55°). The longitudinal shear force and transverse force will give added stresses in the braces, and therefore it is important to select the correct θ which theoretically has to be somewhat above the heading giving maximum longshear force. It may therefore be necessary to evaluate more than one heading.

In addition to the global response, local drag and inertia force on the bracing structure (twin pontoon units) should be accounted for when relevant.

For a ring pontoon unit, this response will give maximum responses at the pontoon/node/column intersections and at the column to deck connections. For a typical ring pontoon unit, the longitudinal shear force will, at areas with maximum long-shear responses, give larger responses than split forces and torsion moment.

Also for ring pontoon units, the maximum responses from simultaneous (longitudinal/transversal) shear forces and split forces should be accounted for. The maximum response is sensitive to the choice of wave heading.

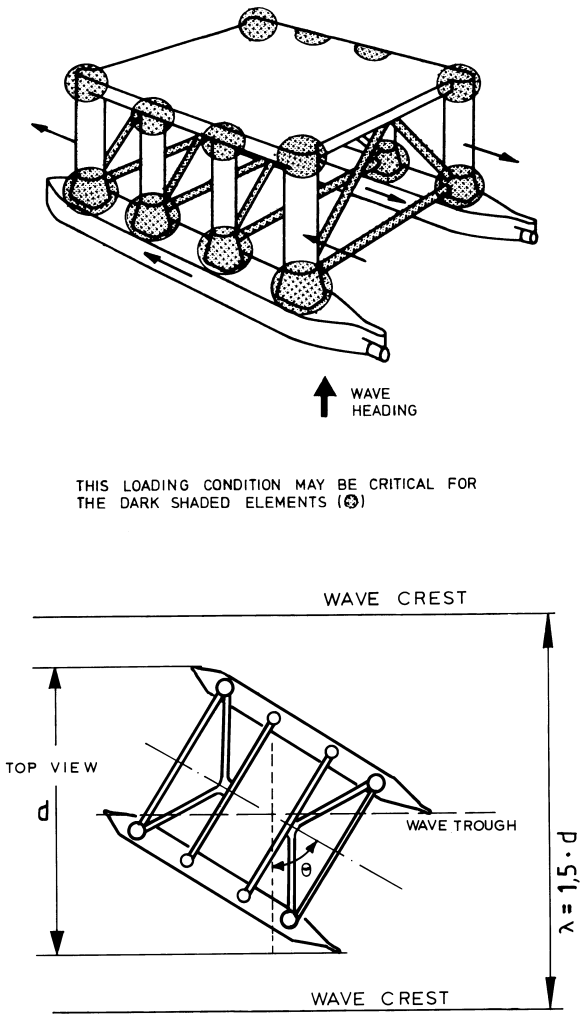


Figure 4-9
Longitudinal shear force

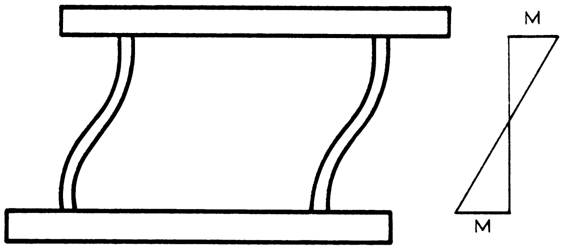


Figure 4-10
Longitudinal deflections (S-moment)

4.6.5 Longitudinal acceleration of deck mass

The critical value for this response will occur at head seas. Typical maximum values are 0.2-0.25 g in survival and 0.1-0.15 g in operation and transit condition.

This response will introduce longitudinal racking due to acceleration of the mass in the deck and associated area.

The longitudinal acceleration of deck mass will introduce shear force and corresponding bending moments for the columns connecting the upper and lower hulls (deck and pontoon), see Figure 4-11.

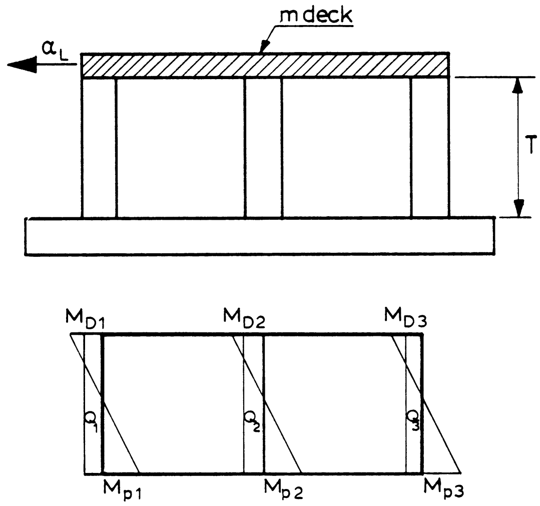


Figure 4-11
Longitudinal racking

4.6.6 Transverse acceleration of deck mass

The critical value for this response will occur at beam seas. Normally this value will reach its maximum at small draughts and may consequently be critical for the choice of operational limits of the transit condition. Typical values are 0.15-0.2 g in survival, 0.1-0.15 g in operation and 0.2-0.25 g in transit condition. This response will introduce transverse racking due to transverse acceleration of the mass in the deck and associated area, see Figure 4-12.

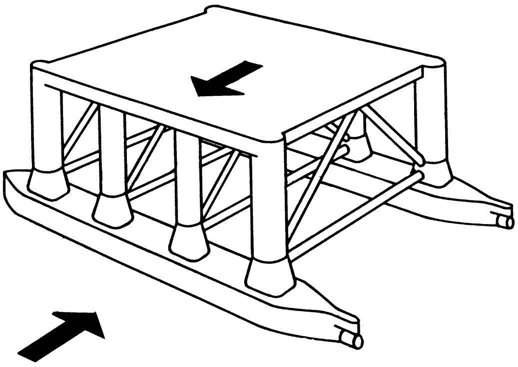


Figure 4-12
Transverse acceleration

This shear force between upper and lower hull (deck and pontoon) has to pass through the construction in one of two ways, see Figure 4-13:

a) For units with diagonal braces the shear force will be experienced as axial force in the braces and shear force with corresponding bending moments in the columns. The distribution between these responses is depending on the stiffness properties. For typical twin pontoon units the response in the columns is rather small and can be neglected compared to the axial response of the diagonal braces.

b) For units without diagonal braces the approach will be similar as presented in 4.6.5.

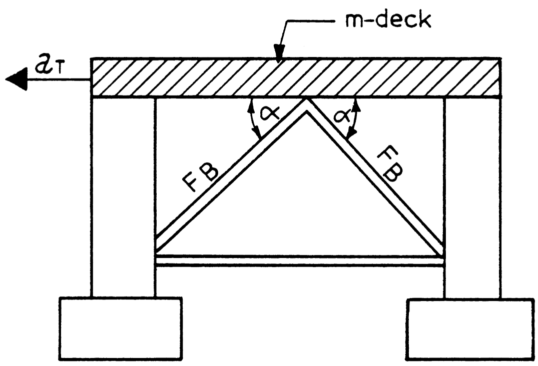


Figure 4-13
Transverse racking

4.6.7 Vertical acceleration of deck mass

This response is in most cases not critical for any global structural element in submerged conditions. The vertical acceleration is illustrated in Figure 4-14. Typical maximum values in survival condition are 0.2-0.25 g.

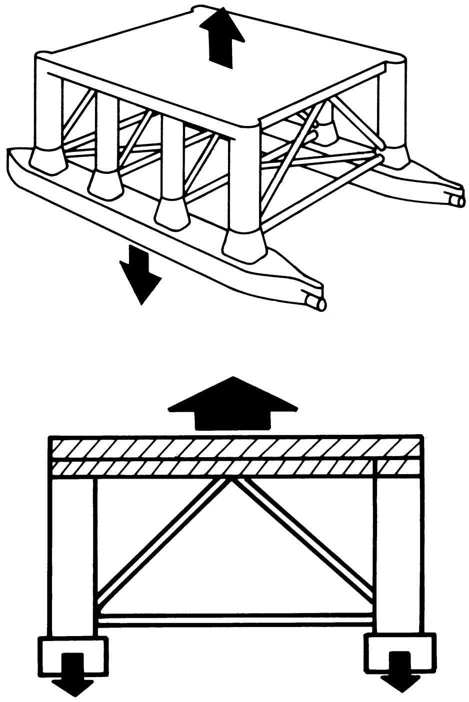


Figure 4-14
Vertical acceleration

4.6.8 Vertical wave bending moment on the pontoon

This response will reach its maximum value at head seas, θ = 0°. The critical wave length will be slightly larger than the pontoon length. The wave height may be derived from the steepness relations given in 2.2.

The wave bending moment should be established with the wave crest at mid pontoon, resulting in a symmetric wave bending moment, see Figure 4-15.

For pontoons with three or more columns, the wave bending moment should in addition be established with the wave zero-crossing point in mid pontoon, giving an asymmetric wave bending moment of the pontoon.

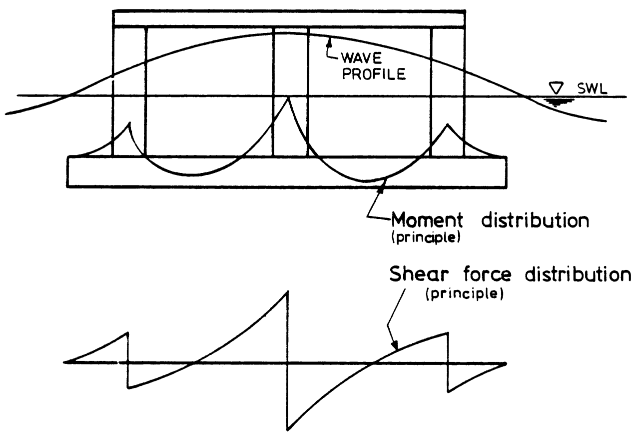


Figure 4-15
Vertical wave bending moment

4.7 Characteristic global static responses

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. Limiting global mass distribution criteria should be established taking into account compliance with the requirements to intact and damage hydrostatic and hydrodynamic stability.

For the pontoons the response is experienced as a bending moment due to the buoyancy in the pontoon, relevant both for twin pontoon units and ring pontoon units. Effect of different ballast distribution shall be adequately accounted for as part of the global load effects for the structural design. Simple frame analyses with beam elements representing pontoons and columns with fixed boundary conditions at deck level may be applied for assessment of different ballast distribution. Maximum and minimum ballast conditions, representing unfavourable conditions used in the tank load plans, are typically given in Figure 4-16. Effects with non-symmetric, diagonally distributed tank loading conditions should also be assessed.

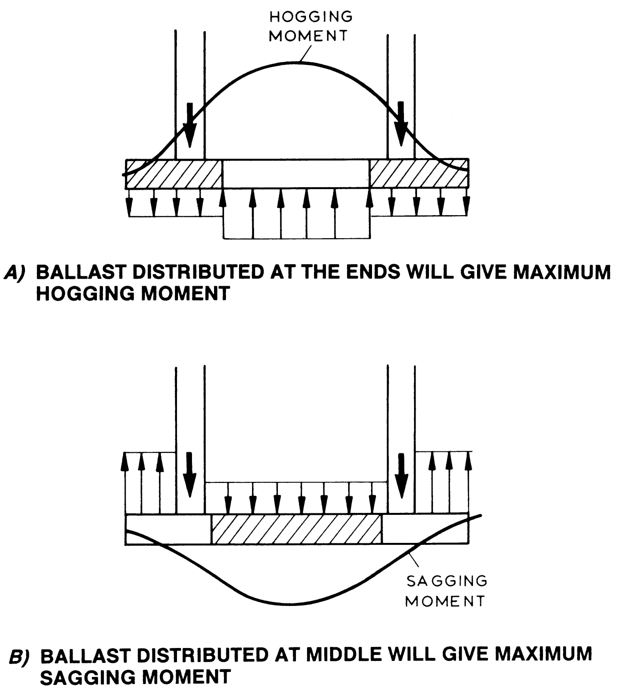


Figure 4-16
Ballast distributions in pontoon

4.8 Global structural model

4.8.1 Types of global structural model

The intention of the global analysis model(s) should be to enable the assessment of responses resulting from global loads.

A global structural model shall represent the global stiffness and should be represented by a large volume, thin-walled three-dimensional finite element model. A thin-walled model should be modelled in shell (or membrane) finite elements, sometimes in combination with beam elements.

Assessment of single model solutions and responses normally not covered in global models are given in 4.8.2.

The structural connections in the model shall be modelled with adequate stiffness (e.g. sufficient detail of connections as pontoon/column and column/deck) in order to represent the actual stiffness such that the resulting global responses are appropriate for the design. Local analyses of such connections may also be required.

Three types of global structural model are referred to in DNV-OS-C103 Appendix B (with Types 3, 4, and 5 as used in Appendix B):

— *Type 3: Beam model.*

  A complete three-dimensional structural beam model is normally not accepted for global structural analysis except in special cases for twin pontoon units in benign waters, following normal class survey intervals.

— *Type 4: Combined shell/beam model.*

  For twin pontoon units with braces between pontoons and deck, such braces may be represented by beam elements. In such cases, local analyses shall be performed for the brace connections to the hull and deck.

— *Type 5: Complete shell model.*

  A complete shell model shall be applied for all ring pontoon units, both hull and deck. Plate stiffeners and girders may be lumped and included as beam elements for correct stiffness representation in the shell model.

The global structural model usually comprises:

— pontoon shell, longitudinal and transverse bulkheads
— column shell, decks, bulkheads and trunk walls
— main bulkheads, frameworks and decks for the deck structure (secondary bulkheads and decks which are not taking part in the global structural capacity should not be modelled)
— bracing structure for twin pontoon units.

Analysis with anisotropic plate stiffness (also referred to as "stressed skin" design philosophy) may in certain cases be applied for the analyses and design of deck structures. Conditions for application and design based on this philosophy are given in 5.2.6.

Examples of global structural models of twin pontoon unit and ring pontoon unit are given in Figure 4-17 and in Figure 4-18, respectively. The twin pontoon model is a combined shell/beam model (Type 4), and the ring pontoon model is a complete shell model (Type 5).

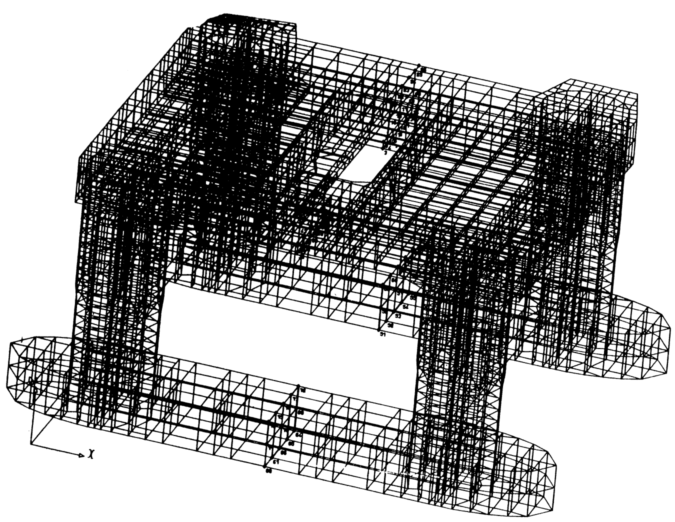


Figure 4-17
Example of twin pontoon global structural model (Type 4 shell/beam model)

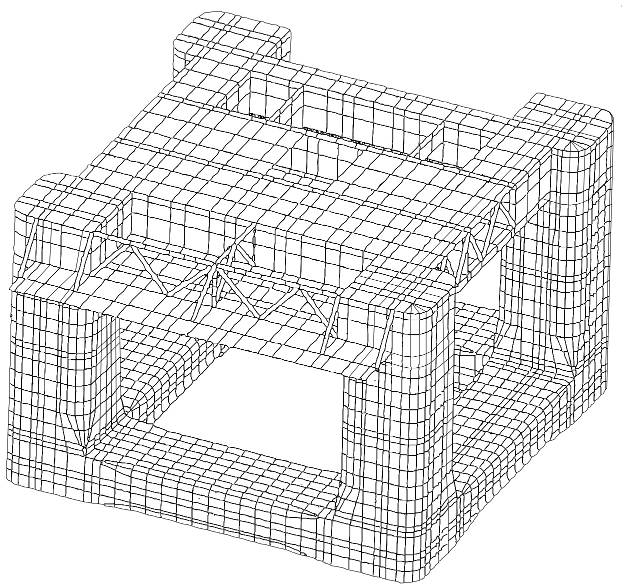


Figure 4-18
Example of ring pontoon global structural model (Type 5 shell model)

4.8.2 Assessment of single model solutions

It is normally not practical to consider all relevant loads (both global and local) in a single model, due to the following reasons:

— Single model solutions do not contain sufficient structural detailing.

  For ULS structural assessment, responses down to the level of the stresses in plate fields between stiffeners are normally required. Examples of insufficient structural detailing may be:

  — internal structure is not modelled in sufficient detail to establish internal structural response to the degree of accuracy required
  — element type, shape or fineness (e.g. mesh size) is insufficient.

— Single model solutions do not normally account for the full range of tank and sea pressure combinations, see 3.8.6.

  Examples of effects that may typically not be fully accounted for include:

  — internal tank pressure up to the maximum design pressure
  — maximum sea pressures (e.g. by use of a "design wave" approach) the sea pressure height resulting from the design wave is not the maximum sea pressure the section may be subjected to
  — variations in tank loading across the section of the pontoon; for example if the pontoon is sub-divided into watertight compartments across its section
  — load conditions that may not be covered by the global structural analysis; for example ALS heeled condition.

— Single model solutions do not normally account for the full range of "global" tank loading conditions, see also 4.7.

Generally, single model solutions that do contain sufficient detail to include consideration of all relevant loads and load combinations are normally extremely large models, with a very large number of loadcases. It is therefore often the case that it is more practical, and efficient, to analyse different load effects utilising a number of appropriate models and superimposing the responses from one model with the responses from another model in order to assess the total utilisation of the structure.

Single model solution with use of the design wave analysis approach as described in 4.3 is not possible to combine.

4.8.3 Mass modelling

A representative number of global design load conditions, simulating the static load distribution for each draught, should be evaluated in the global model. This may be achieved by the inclusion of a "mass model". The mass model may be an independent model or may be implicitly included in the structural (or wave load) model(s).

Usually, only a limited number of load conditions are considered in the global analysis. Therefore the global model may not adequately cover all "worst case" global load distributions for each individual structural element. Procedures shall be established to ensure that the most unfavourable load combinations have been accounted for in the design, see also 3.2.

In respect to global pontoon tank loading arrangements the maximum range of responses resulting from the most onerous, relevant, static load conditions shall be established. In order to assess the maximum range of stresses resulting from variations in pontoon tank loading conditions a simplified model of the structure may be created. This simplified model may typically be a space frame model of the unit, see 4.7.

4.8.4 Boundary conditions

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

Fixed boundaries or spring stiffness may be applied depending on what is the most appropriate for the structure in question. The selection of the boundary conditions may be as illustrated in Figure 4-19, with the following restraints:

— 3 vertical restraints (Z)
— 2 transversal horizontal restraints (Y)
— 1 longitudinal horizontal restraint (X).

When spring stiffness for the vertical restraints are applied, the total vertical stiffness should be according to the water plane area.

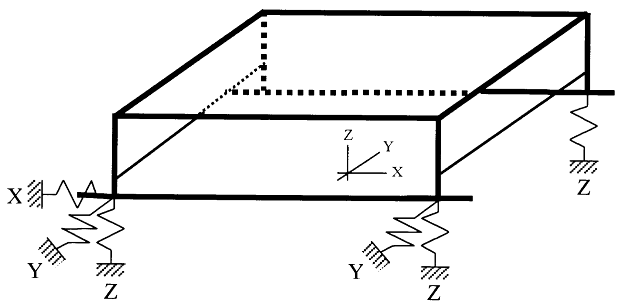


Figure 4-19
Boundary conditions

5. Local Structural Analyses and Strength Criteria

5.1 Introduction

An adequate number of local structural models should be created in order to evaluate response of the structure to variations in local loads, for example in order to evaluate different tank and sea pressure combinations, lay-down loads acting on deck plate field, support loads of heavy equipment/items, stress concentration details for fatigue assessments, etc.

The model(s) should be sufficiently detailed such that resulting responses are obtained to the required degree of accuracy. Several local models may be required in order to fully evaluate local response at all relevant sections.

Design and scantlings shall be performed on basis of strength criteria referred to in DNV-OS-C103 and DNV-OS-C101, based on relevant combination of global and local load effects for each individual structural element, including various tank and sea pressures acting on pontoon and column sections. Guidance for superimposing responses are given in 5.3. Strength criteria for structural utilisation are referred to in 5.4.

5.2 Local structural analyses

5.2.1 General

Four typical modelling levels for analyses of a column-stabilised unit are described. The finite element modelling of the unit or structural component should be carried out in accordance with the principles and details given in the following. Other equivalent modelling procedures may also be applied.

Examples and guidance for local structural models are given in sections 5.2.2 to 5.2.7. It should be noted that the model levels might comprise a wide range of different models and sub-levels of models. A local structural model may be included in higher level model or run separately with prescribed boundary deformations or boundary forces.

Model level 1- Global structural model

As part of the global response analysis, relatively coarse element mesh model for the entire unit should be used. The overall stiffness of the main load bearing members of the hull and deck shall be reflected in the model.

The intention of the global analysis model should be to enable the assessment of responses resulting from global loads.

The model should be used for analysing global wave responses and still water responses where found relevant. See Chapter 4 for assessment of structural components, loads and effects to be included and considered as part of the global structural response analysis. Note the assessment of single model solutions in 4.8.2.

The global structural model may be used for redundancy analyses (ALS), for example for twin pontoon units with ineffective brace(s), see 5.2.7.

Model level 2 - Girder model

The purpose of the local structural analysis with girder model is to analyse structural details (e.g. transverse girders of pontoon or horizontal stringer of hull column) and loading conditions (e.g. different combinations of sea and tank pressures) which have not been accounted for in the global analysis.

The extent of the structural model should be decided based on structural arrangements, loading conditions and method of response application. Typical girder model may be:

— frame model representing girder/stringer and effective plating (beam elements)
— three dimensional shell/membrane model with one or several girders/stringers, plating, web frames, major brackets, and stiffeners (e.g. as bar/beam elements).

The following typical areas should be given particular attention:

— transverse girders and bulkheads in pontoon
— stringers and decks in column
— effect of different tank and sea pressure combinations, see 3.8
— deck areas with concentrated or distributed loads.

Model level 3 - Stiffener between girder model

The purpose of the local structural analysis with stiffener model is to analyse heavy loaded stiffeners and laterally loaded stiffeners, including brackets, subject to relative deformations between girders/stringers.

The following typical areas should be given particular attention:

— longitudinal stiffeners between transverse bulkhead and the first frame at each side at of the bulkhead (pontoon)
— vertical stiffeners between horizontal decks and stringer in column
— stiffeners at bulkheads in way of pontoon-column and column-deck intersections
— for twin pontoon units: brace to column connection and brace to deck connection
— local support areas for support of for example fairleads, windlass, crane pedestal, drilling derrick, flare tower, living quarters, etc.
— effect of different tank and sea pressure combinations, see 3.8.

Model level 4 - Stress concentration models

For fatigue assessment, fine element mesh models should be made for critical stress concentration details, for details not sufficiency covered by stress concentration factors (SCF) given in recognised standards, see for example DNV-RP-C203.

The following typical areas should be given particular attention:

— hot spot stress at the cruciform plate connections in way of pontoon-column and column-deck intersections
— hot spot stress in the welded supports of for example fairleads, crane pedestal, flare tower, etc.
— hot spot stress at local column/brace connection (twin pontoon)
— hot spot stress at attachments
— details in way of the moonpool
— large and small penetrations
— corners at door openings
— stiffener and girder terminations
— weld profiling of cruciform joints
— cast insert pieces.

The size of the model should be of such extent that the calculated stresses in the hot spots are not significantly affected by the assumptions made for the boundary conditions.

Element size for stress concentration analyses is normally to be in the order of the plate thickness. Normally, 8-node shell elements or 20-node solid elements should be used for the analysis. The correlation between different loads such as global bending, external and internal fluid pressure and acceleration of the topside should be considered in the fatigue assessment. For further details see DNV-RP-C203 item 2.13.

In some cases detailed element mesh models may be necessary for ultimate limit state assessment in order to check maximum peak stresses and the possibility of repeated yielding.

5.2.2 Pontoons

The pontoon may be divided into the following structural elements:

— pontoon top, sides and bottom
— longitudinal bulkheads
— transverse bulkheads with girders
— transverse frames and longitudinal girders.

Example of a local analysis model of part of pontoon structure is given in Figure 5-1.

Simple frame (beam element) or shell models may also be applied to for example transverse girders.

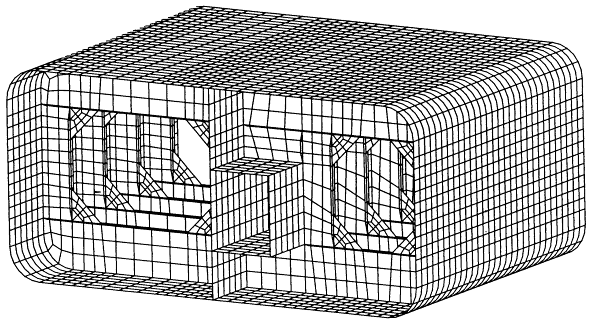


Figure 5-1
Part of ring pontoon local analysis model (model level 2)

5.2.3 Columns

The columns may be divided into the following structural elements:

— shell plating and stiffeners
— horizontal stringers
— horizontal decks
— vertical bulkheads.

Local models may be of similar types as applied to the pontoons.

For local strength analysis in way of fairlead and windlass, the design should be based upon the breaking strength of anchor lines, see also 6.1:

— local support of windlass
— local support of fairlead
— load effects on column shell between fairleads and anchor winches.

The load effects on column shell between fairleads and anchor winches may be estimated by hand calculations, applying relevant width of the external shell plating of the column as load carrying area, and combined with the stresses from the global response analysis.

5.2.4 Pontoon/column and column/deck intersections

Local design of both pontoon/column and column/deck intersections is normally governed by the fatigue criteria, with local peaks of excessive yielding and buckling.

In particular the major intersections of pontoon/column of a typical ring pontoon unit are sensitive to fatigue fracture, requiring special attention to local analyses and design at the following positions:

— centre bulkhead pontoon/column intersection at pontoon upper deck
— pontoon outer wall/column intersection at pontoon upper deck
— pontoon/pontoon intersections.

5.2.5 Brace to column connection (twin pontoon units)

Requirements for brace arrangements are given in DNV-OS-C103 Sec.7 A200. The forces in the brace structures shall be transmitted to the surrounding structure. The connections are normally determined by fatigue of hot spots and yielding.

The following considerations are recommended for the structural design and local analysis of brace to column connections:

— brace forces should be transmitted through sound connections to bulkheads, decks and shell (circular connections)
— support flats should be checked for buckling with actual in plane and bending stresses and hydrostatic pressure if present (tank pressure)
— shear capacity of the support flats should exceed the axial capacity of the brace
— effect of bending moment in the brace should be accounted for.

A local analysis model for fatigue assessment of brace to column connection is shown as example in Figure 5-2.

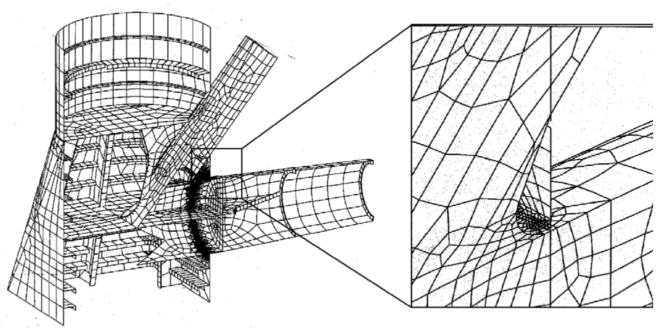


Figure 5-2
Part of detailed shell element model of column/brace connection

5.2.6 Deck structure

Figure 5-3 shows a typical part of deck structure, with the deck divided into the following structural categories:

— primary girders/bulkheads
— secondary girders
— stiffeners
— plate.

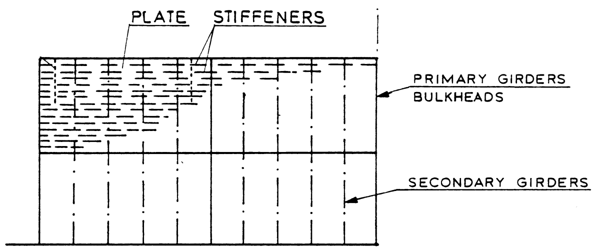


Figure 5-3
Typical part of deck structure

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.

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.

Fatigue

Fatigue evaluation of the deck structure shall be performed. Such evaluation may be based on a screening process with a simplified fatigue analysis approach to identify critical areas/ details. Special attention should be made to the following:

— areas of the deck where the dynamic stresses are significant
— areas of the deck where the stress concentration factor may become large, such as at penetrations, cut-outs, door openings, attachments, flare supports, crane supports, etc.

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)).

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

5.2.7 Damaged structure

Requirements for Accidental Limit States (ALS) and structural redundancy of slender main load bearing structural elements are given in DNV-OS-C103 Sec.6 and DNV-OS-C103 Sec.7.

The damaged condition may be divided in two main groups:

1) *Structural redundancy*

   comprising fracture of bracing or joint between bracings (for twin pontoon units) and fracture of primary girder/ truss element in deck structure.

2) *Stability and watertight integrity*

   see intact and damage stability requirements including compartmentation and watertight integrity in DNV-OS-C301.

   The structural design comprise flooding of tanks and void spaces with heel angle after loss of buoyancy not to exceed 17°.

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.

Fracture of bracing or joint (for twin pontoon units)

Analyses for static and wave induced loads should be carried out for damaged cases assuming successive bracings to be ineffective. The global analysis model may normally be used. Typical damaged case are illustrated in Figure 5-4.

If the 100 year return period is used as basis for the analyses, the 1 year responses may taken as:

$$\sigma_1 = 0.77^{1/h}\sigma_{100}$$

σ_1 = one year stress response
σ_{100} = 100 year stress response
h = Weibull shape parameter; a value of 1.1 may be applied together with-worldwide criteria for twin pontoon units if not further documented.

Local yield and buckling can be accepted provided it can be demonstrated that excessive forces can be redistributed to other members. Such redistribution may be demonstrated by different methods:

— recalculation with reduced stiffness of elements with plastic behaviour
— redistribution by hand calculation of the excessive forces obtained as the difference between the analysed forces in the elastic analysis and the plastic capacity.

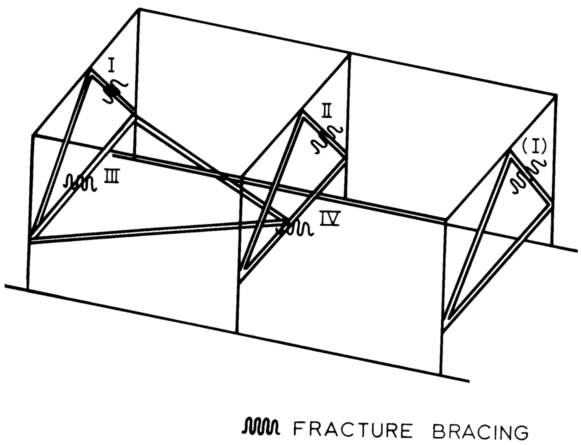


Figure 5-4
Example of damaged cases for redundancy analyses

Fracture of primary girder/truss element in the deck structure

The structural arrangement of the deck structure with main girders/truss elements is to be considered with regard to the structural integrity after failure of any primary girder/truss element, similar as described above for fracture of bracing or joint.

Fires or explosions may be the critical event for this accidental scenario.

Heeled condition after damage flooding

As part of the damaged stability requirement, see DNV-OS-C301, the static heel angle is not to exceed 17° in any direction.

The structural design comprises flooding of tanks and void spaces. The hydrostatic pressure in flooded spaces is the vertical distance between a load point and damaged waterline in static heeled condition (17°), see 3.8.5.

At this 17° heel angle the gravity component parallel to the deck is 0.29 g. This static component and the dynamic effects shall be accounted for in the assessment of structural strength of deck structure. Local exceedance of the structural resistance is acceptable provided redistribution of forces due to yielding, buckling and fracture is accounted for.

In the damaged heeled condition (e.g. ALS condition after a collision event), the unit shall resist the defined environmental conditions corresponding to 1-year return period. It is not normally considered practicable to analyse the global structure in this damaged, inclined condition with wave loads as the deck structure becomes buoyant, due both to the static angle of inclination and also due to rigid body motion of the unit itself. The global system of loading and response becomes extremely non-linear. Additionally, as soon as the deck structure starts to become buoyant, the global load effects resulting from the inclined deck mass rapidly become reduced. Hence, it is normally acceptable not to perform global response analysis with wave loads for the 17^{o} heeled condition. The effects of environmental loads may be accounted for by use of the material factor γ_M =1.33 as given in DNV-OS-C103 Sec.6 F102.

5.3 Superimposing responses

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 several different models, consideration shall be given to the following:

- Loads applied in global and local models as discussed in 3.2.
- Relevant combination of tank and sea pressures as discussed and specified in 3.8.6.
- Assessment of different global and local model solutions as discussed in 4.8.2.
- It should be ensured that responses from design loads are not included more than once (see for example the effect of simple frame analysis models in sections 4.7 and 4.8.3 related to variations in pontoon tank loading conditions).
- Continuous, longitudinal structural elements, for example stiffened plate fields in the pontoon deck, bottom, sides, bulkheads, tunnels etc., located outside areas of global stress concentrations, may be evaluated utilising linear superposition of the individual responses as illustrated in Figure 5-5 for a pontoon section.
- When transverse stress components are taken directly from the local structural model ($\sigma_{y\ (Local\ Model)}$) in Figure 5-5), the transverse stresses from the global model may normally be neglected.
- Stiffener induced buckling failure normally tends to occur with lateral pressure on the stiffener side of the plate field. Plate induced buckling failure normally tends to occur with lateral pressure on the plate side. Relevant combinations of buckling code checking should therefore include evaluation of the capacity with relevant lateral pressure applied independently to both sides of the plate field.
- In order to ensure that local bending stress components resulting from loads acting directly on the stiffeners are included in the buckling code check, the lateral pressure should be explicitly included in the capacity check. The capacity checking should include a buckling check with no lateral pressure in addition to the case with lateral pressure (unless there is *always* pressure acting over the stiffened plate field being evaluated).
- Superimpose local compression stresses from bending of deck girders (stiffeners) with global compression stresses in buckling check as described in 5.2.6.

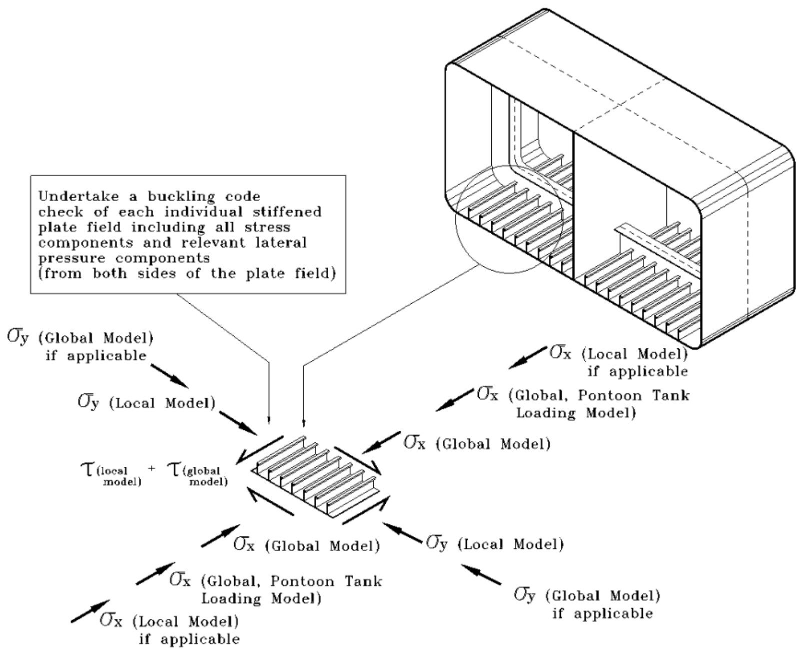


Figure 5-5
Combination of stress components for buckling assessment of an individual stiffened plate field in a typical pontoon section

5.4 Strength criteria

5.4.1 General

Structural utilisation shall be evaluated in accordance with the requirements of the limit states ULS, FLS, and ALS as referred to in DNV-OS-C103 and DNV-OS-C101, comprising the following modes of failure to be considered:

— excessive yielding (ULS, ALS)
— buckling (ULS, ALS)
— brittle fracture
— fatigue fracture (FLS).

5.4.2 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.

5.4.3 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.

5.4.4 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-C103 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.

5.4.5 Fatigue

Requirements to the Fatigue Limit States (FLS) are given in DNV-OS-C103 Sec.5 and DNV-OS-C103 Appendix A and DNV-OS-C103 Appendix B, and in DNV-OS-C101 Sec.6.

Guidance concerning fatigue calculations are given in DNV-RP-C203.

Design Fatigue Factors (DFF) shall be applied as fatigue safety factors for permanently installed units, as specified in DNV-OS-C103 Sec.5 A103 and DNV-OS-C103 Appendix A. The applications of DFFs are also discussed in DNV-OS-C101 Sec.6. The calculated fatigue life shall be longer than the design fatigue life times the DFF.

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. 10^8 cycles) may be taken as:

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

$\Delta\sigma_{20}$ = extreme stress range during 20 years (10^8 cycles)
$\Delta\sigma_{100}$ = extreme stress range during 100 years ($10^{8.7}$ cycles)
h = Weibull stress range shape distribution parameter; a value of 1.1 may be applied together with world-wide criteria for twin pontoon units if not further documented.

Guidance for simplified fatigue analysis are given in DNV-OS-C103 Sec.5 B400 and in DNV-RP-C203 item 2.14. 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.

Use of one slope S-N curves leads to results on the safe side.

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 (10^8 cycles). Note that the allowable extreme stress ranges should be reduced for longer design fatigue lives, DFFs and thickness effects.

6. Miscellaneous

6.1 Support of mooring equipment

6.1.1 General

Requirements for the position mooring system are given in DNV-OS-E301. The following items relate directly to the mooring lines and the mooring equipment (windlass/winch, chain stopper, fairlead) supported on the hull and deck structure of the unit:

— structural design procedure for the mooring lines, including mooring system analysis and design criteria formulated in terms of the limit states ULS, ALS, and FLS, are specified in DNV-OS-E301 Ch.2 Sec.2
— recommendations and methods for the design of thruster assisted moorings are specified in DNV-OS-E301 Ch.2 Sec.3
— structural design procedure for the mooring equipment such as windlass/winch, chain stopper, and fairlead are specified in DNV-OS-E301 Ch.2 Sec.4. The design of these components is based on a load equal to the characteristic breaking strength of the mooring lines.

As specified in DNV-OS-C103 Sec.7 B100, the local structure in way of fairleads, winches, etc. forming part of the fixed position mooring system, shall withstand forces equivalent to 1.25 times the breaking strength of any individual mooring line. The strength evaluation should be undertaken utilising the most unfavourable operational direction of the anchor line. In the evaluation of the most unfavourable direction, account shall be taken of relative angular motion of the unit in addition to possible line lead directions.

The above referred load factor of 1.25 (applied on the breaking strength of mooring line) can be regarded as similar safety factor as specified in DNV-OS-E301 Ch.2 Sec.4 N109, with maximum utilisation of 0.8 times yield strength of the supporting structures.

In addition all structural elements influenced by the mooring loads shall be designed to have sufficient strength to withstand relevant loads acting on the mooring system as described for the ULS and the ALS.

6.1.2 Design loads

The supporting structure influenced by the mooring forces, such as support of winches and fairleads and the column shell between winch and fairlead, shall be designed for the two following main loading conditions:

a) Breaking load of one single mooring line:

$$F_{d,\,w1} = F_B \gamma_f$$

$F_{d,w1}$ = design load on windlass (corresponding to one mooring line)
F_B = characteristic breaking strength of one mooring line
γ_f = 1.25 (load factor, see 6.1.1)

The material factor γ_M is 1.0 in this case.

b) Operational loads from all mooring lines:

The design of all structural elements influenced by the mooring loads shall take into account relevant loads (ULS and ALS) found from the mooring analysis.

The static and dynamic contributions to the mooring line forces should be considered for relevant application of load and material factors according to DNV-OS-C103 Sec.4 (ULS) and sec. 6 (ALS).

6.1.3 Horizontal and vertical design angles for the fairlead supports

Fairlead with vertical inlet angle and horizontal working angle is shown in Figure 6-1, with mooring line tension T.

The most critical vertical inlet angle γ and horizontal working angle φ shall be considered for the local strength analysis and design of the fairlead supporting structure:

— The vertical design inlet angle γ_d should not be taken larger than 10° unless otherwise documented (the support resultant force on supporting structure will decrease with increased γ).
— The horizontal design working range (DWR) should be 20° larger than the operational working range (2 φ), see DNV-OS-E301 Ch.2 Sec.4 L300:

DWR = 2 φ + 20°

.

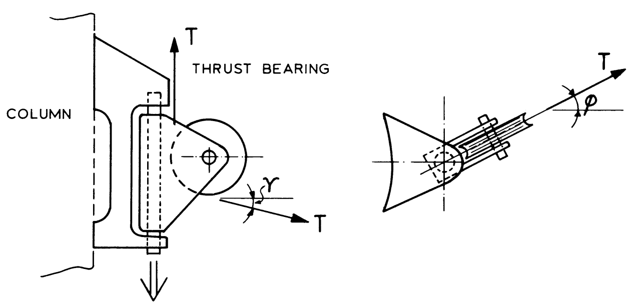


Figure 6-1
Fairlead with vertical inlet angle (γ) and horizontal working angle (φ)

6.2 Wave slamming

Slamming on bracing structures on typical twin pontoon units may be a governing criterion for deciding when to submerge from transit draught. It is normally stated as a design criterion that significant slamming should be avoided. For design stage evaluation of the transit capabilities the following criteria may be used:

Slamming frequency: The number of slamming impacts should not exceed 1 slam per 20 wave encounters.

Extreme stress: The extreme slamming induced stress in the brace in a certain stationary design period (e.g. 3 hours) should satisfy ULS combination b).

Fatigue rate: The fatigue growth rate due to slamming in transit condition should not be greater than the nominal average rate for the life time of a structural element.

Slamming is defined as a situation occurring when a structural member (brace) hits the water surface due to relative motion between the unit and the waves. Two conditions must be fulfilled to get slamming:

— The relative vertical motion must exceed the distance from the exposed member to the still water surface.
— The velocity of the member relative to the wave must be greater than a certain threshold value. This value is normally defined when the slamming force becomes greater than the buoyancy force (guidance 3-3.5 m/s).

By taking into account the probability for exceeding the above mentioned conditions, number of slamming and most probable largest slamming force per unit length during N wave cycles may be calculated. The above mentioned slamming criteria may be calculated for different sea states (H_s, T_z) and speeds. It may be possible to specify limiting sea states for when it is necessary to submerge from transit draught or to slow down forward speed.

Slamming loads from waves and procedure for fatigue damage are given in Classification Note 30.5 6.4.

6.3 Air gap

Requirements for sufficient air gap are specified in DNV-OS-C103 Sec.4 D100.

In the ULS condition, positive clearance between the deck structure and the wave crest, including relative motion and interaction effects, should normally be ensured. Localised, negative air gap may be considered as being acceptable for overhanging structures and appendages to the deck structure. In such cases full account of the wave impact forces is to be taken into account in the design. The consequence of wave impact shall not result in failure of a safety related system (e.g. lifeboat arrangements).

The wave asymmetry factor in air gap calculations is given in 2.3.6.

It is recommended, in the design phase, to consider operational aspects, including requirements to inspection and maintenance, which may impose criteria to air gap that exceed minimum requirements.

In the context of DNV-OS-C103 Sec.4 D103, column run-up load effects are not considered as resulting in negative air-gap responses.

6.4 Transit condition

6.4.1 General

Weather restrictions and criteria related to the transit condition shall include consideration of the following items:

1) Motions and accelerations; see sections 6.4.2 and 6.4.3 below
2) Wave slamming, see 6.2
3) Stability during the ballasting sequences, see DNV-OS-C301.

6.4.2 Transit analysis

To decide the critical sea states when it is necessary to submerge a column-stabilised unit from transit draught, the design responses as defined in 4.6.1 may be calculated in transit condition (strip theory or 3-D diffraction theory), and the maximum responses taken equal to the values obtained for the survival/operating conditions in a 100 year storm condition.

An example is shown in Figure 6-2. From such chart it may be possible to conclude which responses that may be critical for the transit condition as well as limiting environmental criteria.

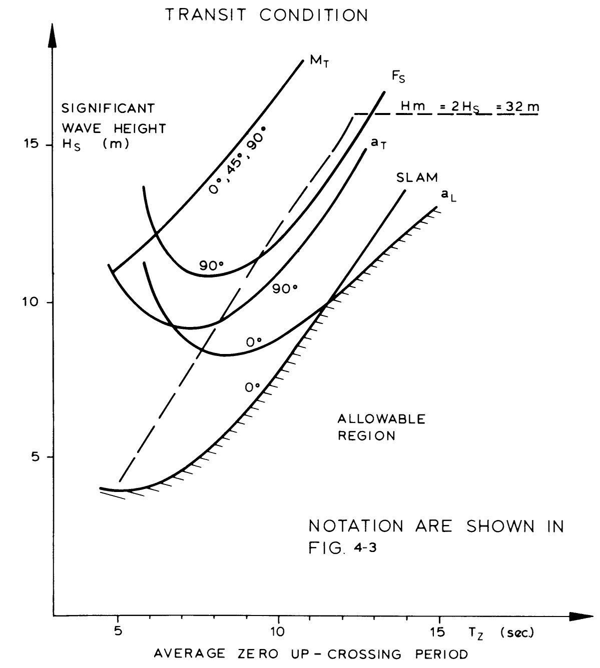


Figure 6-2
Critical sea states for transit condition (example)

Direct calculations of the transit analysis may be an alternative approach, with similar wave load and stress analysis approach as for the operating and survival conditions.

6.4.3 Wave loads

For transit condition the strip theory or the more accurate 3-D diffraction theory should be used as the effect of the free surface is significant. Forward speed may normally be neglected as the maximum forces occur for zero speed, except for local slamming (see 6.2).

Flow of water on top of the pontoons will introduce non-linear effects that are not taken into account in the linear calculations. Experience from model tests indicates that the primary results of these non-linearities are to reduce the heave, pitch and roll motions near resonance.

A normally acceptable procedure is to reduce the resonance amplitudes of these motions to approximately 10% above their asymptotic values. The asymptotic values are:

$$\text{Heave:} \quad \frac{MO}{a} = 1.0$$

$$\text{Roll and Pitch:} \quad \frac{\theta}{a/\lambda} = 2\pi$$

MO = motion amplitude
a = wave amplitude
θ = rotation amplitude in radians
λ = wave length

Surge should be corrected due to coupling with pitch. Normally acceptable multiplication factor for the linear motion of the resonance peak is 0.5.

Sway and yaw are not so strongly affected by the linearities and no corrections should be made.