

Actions and action effects

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Foreword

The NORSOK standards are developed by the Norwegian petroleum industry to ensure adequate safety, value adding and cost effectiveness for petroleum industry developments and operations. Furthermore, NORSOK standards are, as far as possible, intended to replace oil company specifications and serve as references in the authorities' regulations.

The NORSOK standards are normally based on recognised international standards, adding the provisions deemed necessary to fill the broad needs of the Norwegian petroleum industry. Where relevant, NORSOK standards will be used to provide the Norwegian industry input to the international standardisation process. Subject to development and publication of international standards, the relevant NORSOK standard will be withdrawn.

The NORSOK standards are developed according to the consensus principle generally applicable for most standards work and according to established procedures defined in NORSOK A-001.

The NORSOK standards are prepared and published with support by The Norwegian Oil Industry Association (OLF), The Federation of Norwegian Industry, Norwegian Shipowners' Association and The Petroleum Safety Authority Norway.

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Introduction

The principal standard for offshore structures is NORSOK N-001, *Structural design*, which especially refers to ISO 19900, *Petroleum and natural gas industries - General requirements for offshore structures*.

It is the intention to revise this NORSOK standard as soon as the International Standards covering the scope of this NORSOK standard have been published.

1 Scope

This NORSOK standard specifies general principles and guidelines for determination of actions and action effects for the structural design and the design verification of structures.

This NORSOK standard is applicable to all types of offshore structures used in the petroleum activities, including bottom-founded structures as well as floating structures.

This NORSOK standard is applicable to the design of complete structures including substructures, topside structures, vessel hulls, foundations, mooring systems, risers and subsea installations.

This NORSOK standard is applicable to the different stages of construction (namely fabrication, transportation and installation), to the use of the structure during its intended life, and to its abandonment. Aspects related to verification and quality control are also addressed.

2 Normative and informative references

2.1 Normative references

The following standards include provisions and guidelines which, through reference in this text, constitute provisions and guidelines of this NORSOK standard. Latest issue of the references shall be used unless otherwise agreed. Other recognized standards may be used provided it can be shown that they meet the requirements of the referenced standards.

API RP 2A-LRFD,	Planning, Designing and Constructing Fixed Offshore Platforms - Load and Resistance Factor Design, Clause 17
Det Norske Veritas,	Classification note No. 30.5 Environmental conditions and Environmental loads, Oslo 2000
ISO 2103:1986,	NOTE DNV CN 30.5 will be replaced by DNV RP C205 in 2007
ISO 19901-1,	Loads due to use and occupancy in residential and public buildings
ISO/DIS 19902,	Petroleum and natural gas industries – Specific requirements for offshore structures – Part 1: Motoccean design and operating considerations
NCAA Regulations,	Petroleum and natural gas industries – Fixed steel offshore structures
	Regulations relating to commercial aviation to and from helicopter decks on fixed and mobile offshore installations, Bestemmelser for sivil luftfart (BSL) D5-1, Norwegian Civil Aviation Authority
NMD Regulations No. 856,	Regulations of 4 September 1987 No. 856 concerning construction of mobile offshore structures: issued with amendments 1997)
NMD Regulations No. 857,	Regulations of 4 September 1987 No. 857 concerning anchoring/positioning on mobile offshore units (issued with amendments 1997)
NMD Regulations No. 878,	Regulations of 20 December 1991 No. 878 concerning stability, watertight subdivision and water tight/weather tight closing means on mobile offshore units
NMD Regulations No. 123,	Regulations of 10 February 1994 No. 123 for mobile offshore units with production plants and equipment
NORSOK N-001,	Structural design
Sarpkaya T and Isaacson M.,	Mechanics of wave forces on offshore structures, New York (1981)

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API RP 2N,	Planning, Designing and Constructing Structures and Pipelines for Arctic Conditions (1995)
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DNV-OS-E301, Det Norske Veritas "Position mooring", (2001)

DNV-OS-F201, Det Norske Veritas, "Dynamic Risers", Oslo (2001)

ENV 1991-2-2, Eurocode 1: Part 2-2: Actions on structures exposed to fire (1995)

ENV 1991-2-4, Eurocode 1: Part 2-4: Wind actions (1995)

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NFR/NORSAR (1998), NFR/NORSAR, 1998. "Seismic Zonation for Norway", Norwegian Council for Building Standardization (NBR), Oslo, 1998

NORSOK N-002, Collection of metocean data

NORSOK J-003, Marine operations

NORSOK N-004, Steel structures

NORSOK S-001, Technical safety

NORSOK U-001, Subsea structures and piping systems

NORSOK Z-013, Risk and Emergency Preparedness Analysis (Rev. 2, 2001)

NS 3491-3, Prosjektering av konstruksjoner – Dimensjonerende laster – Del 3: Snølaster (innbefatter rettelsesblad AC:2002) (Design of structures – Design actions – Part 3: Snow loads (Corrigendum AC-2002 incorporated)

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SCI, 1992-1999: "Interim Guidance Notes for the Design and Protection of Topside Structures against Explosion and Fire", Steel Construction Institute, Document SCI-P-112/503

Supplementary Technical Notes:

- TN1: Fire resistance design of offshore topside structures, 1993
- TN2: Explosion Mitigation Systems, 1994
- TN4: Explosion Resistant Design of Offshore Structures, 1996
- TN5: Design Guide for Stainless Steel Blast Walls, 1999

SNAME (2002), "Guideline for site specific assessment of Mobile Jack-up units", Society of Naval Architects and Marine Engineers (USA)

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3 Terms, definitions, abbreviations and symbols

3.1 Terms and definitions

For the purposes of this NORSOK standard, the following terms, definitions, abbreviations and symbols apply.

3.1.1

can

verbal form used for statements of possibility and capability, whether material, physical or casual.

3.1.2

design premises

set of project specific design data and functional requirements which are not specified or are left open in the general standard.

3.1.3

may

verbal form used to indicate a course of action permissible within the limits of the standard

3.1.4

petroleum activities

offshore drilling, production, treatment and storage of hydrocarbons.

3.1.5

shall

verbal form used to indicate requirements strictly to be followed in order to conform to the standard and from which no deviation is permitted, unless accepted by all involved parties

3.1.6

should

verbal form used to indicate that among several possibilities one is recommended as particularly suitable, without mentioning or excluding others, or that a certain course of action is preferred but not necessarily required

3.1.7

verification

examination to confirm that an activity, a product or a service is in accordance with specified requirements

3.2 Abbreviations

ALS	accidental damage limit state
API	American Petroleum Institute
CQC	complete quadratic combination
DAF	dynamic amplification factor
DIS	draft International Standard
DNV	Det Norske Veritas
DP	dynamic positioning
FE	finite element
FLS	fatigue limit state
HF	high frequency
ISO	International Organisation for Standardisation
KC	Keulegan-Carpenter number

LF	low frequency (lower than wave frequency)
NCAA	Norwegian Civil Aviation Authority
NMD	Norwegian Maritime Directorate
NFR	Norges Forskningsråd (The Research Council of Norway)
NS	Norsk Standard (Norwegian Standard)
P- Δ effect	second order effect of an axial force (P) due to a lateral displacement, Δ
SLS	serviceability limit state
SN	stress range (S) versus number of cycles to failure (N)
SRSS	square root of sum of squares
TLP	tension leg platform
ULS	ultimate limit state
VIV	vortex induced vibrations
WF	wave frequency

3.3 Symbols

A	area
C	coefficient
C	damping matrix
C_D	drag coefficient
C_M	mass coefficient
C_S	shape coefficient for wind action
D	diameter of a cylinder
H	height of a regular wave
H_d	design (regular) wave height
H_s	significant wave height
H_{100}	wave height with annual probability of exceedance of 10^{-2}
$I_n()$	turbulence intensity factor
KC	Keulegan-Carpenter number
K	stiffness matrix
M	mass matrix
N_x	number of zero upcrossing of X
Re	Reynolds number
$S(\omega)$	power spectral density
S	action effect
T	period of a regular wave
T_i	intrinsic wave period
T_p	spectral peak period
X	displacement of the structure
$\dot{X}$	time derivative of X
X_{LF}	displacement induced by low frequency wave actions or dynamic wind
X_{WF}	displacement induced by wave frequency actions
U_0	reference particle velocity in waves or wind speed
U	particle velocity
U_m	mean particle velocity
U_R	relative velocity between structure and fluid
U_{RN}	relative velocity normal to the member
$u(z,t)$	wind speed
f	action reduction factor (defined in Table 1)
$f_X(x)$	probability density function
h_D	dynamic pressure head due to flow through pipes
h_{pc}	vertical distance from the action point to the max filling point
h_s	vertical distance from the action point to the top of the tank
p_o	valve opening pressure
p_i	pressure
p	distributed action
P	point action
t	time
t_r	reference time
z	vertical coordinate (zero at water surface)
z_r	reference value of z
α	angle between the direction of wind and exposed surface
θ	wave direction

θ_m	mean wave direction in a short-term sea state
σ_x	standard deviation of X
ρ	density
ζ	percentage of damping

4 Permanent actions

4.1 General

Permanent actions are actions that will not vary in magnitude, position or direction during the time period considered. Examples are

- a) weight of the structure,
- b) weight of permanent ballast and equipment, including mooring and risers,
- c) external hydrostatic pressure up to the mean water level,
- d) pretension.

4.2 Hydrostatic pressure difference

Structural components subjected to high counteracting hydrostatic pressures, such as external walls with more than 100 m of differential water pressure height, should be designed taking into account the effect of possible variation in level and density, dimension tolerances, measuring inaccuracies and other uncertainties affecting the pressure difference. Unless documented otherwise by detailed analyses of operations, the minimum pressure difference for ultimate limit state should be at least equal to the smallest of one tenth of the maximum pressure and 0,1 MPa.

5 Variable actions

5.1 General

Variable actions originate from normal operation of the structure and vary in position magnitude and direction during the period considered. They include those from

- a) persons,
- b) helicopters,
- c) lifeboats,
- d) cranes,
- e) tank pressures,
- f) stored liquids and goods,
- g) modules and structural parts that can be removed,
- h) boat impact, fendering and mooring,
- i) weight of gas and liquid in process plants,
- j) pressure and temperature,
- k) variable ballast,
- l) installation and drilling operations.

Assumptions regarding variable actions shall be reflected in the operating manual, and complied with in operation. Possible deviations from the assumed value due to operational errors or mechanical failure or damage shall be treated as accidental actions, see clause 8.

5.2 Crane actions

Crane actions shall be determined with due account of dynamic effects due to, and, if applicable, the motions of the installation.

Fatigue calculations shall be carried out based on expected frequency of crane usage, the magnitude of actions, dynamic effects from wind, loading and discharging of ships and if applicable from motions of the installation.

5.3 Deck area actions

Variable actions on deck areas of the topside structure shall be based on Table 1 unless specified otherwise in the design premises. The intensity of the distributed actions depends on local/global aspects of the deck structure as shown in Table 1. The following notations are used:

Local design: design of deck plates and stiffeners
 Primary design: design of deck beams and beam-columns
 Global design: design of deck main structure (and substructure)

Table 1 – Variable actions in deck areas

Area	Local design		Primary design	Global design See NOTE 5
	Distribution action, p (kN/m ²)	Point action, P (kN)	Apply factor given below to distributed action, p	Apply factor given below to distributed action, p
Storage areas	q	$1,5 q$	1,0	1,0
Laydown areas	q	$1,5 q$	f	f
Lifeboat platforms	9,00	9,00	1,0	may be ignored
Area between equipment	5,00	5,00	f	may be ignored
Walkways, staircases and platforms	4,00	4,00	f	may be ignored
Walkways and staircases for inspection and repair only	3,00	3,00	f	may be ignored
Roofs, accessible for inspection and repair only	1,00	2,00	1,0	may be ignored

NOTE 1 Wheel actions to be added to distributed actions, where relevant. Wheel actions can normally be considered acting on an area of 300 mm x 300 mm.

NOTE 2 Point actions to be applied on an area 100 mm x 100 mm, and at the most severe position, but not added to wheel actions or distributed actions.

NOTE 3 q is to be evaluated for each case. Storage areas for cement, wet or dry mud should be the maximum of 13 kN/m² and $\rho g H$, where H is the storage height in m. Laydown areas not normally to be designed for less than 15 kN/m².

NOTE 4 f is the minimum of 1,0 and $(0,5 + 3/A^{0,5})$, where A is the action area in m².

NOTE 5 Global action cases should be established based upon "worst case", representative variable action combinations, complying with the limiting global criteria to the structure. For buoyant structures these criteria are established by requirements to the floating position in still water and intact and damage stability requirements, as documented in the operational manual, considering variable actions on the deck and in tanks.

For actions on floors in accommodation and office sections, see ISO 2103:1986.

Guard rails should be designated for the following actions:

- a horizontal line action of 1,5 kN/m acting on the handrail;
- a point action of 1,0 kN acting in worst location and worst direction (horizontal or vertical). The point action does not act together with the horizontal line action;
- actions from possible attachments shall be established for each case;
- actions on guard rails in areas with cargo handling should be determined with due account of relevant operational conditions.

5.4 Tank pressures and tank weight

5.4.1 Hydrostatic pressures

Hydrostatic pressures in tanks should normally be based on a minimum density equal to that of sea water. For tanks dedicated to fluids with a lower density, the actual density may be used. Tanks for heavier fluids (e.g. mud) should be based on special consideration.

The density upon which the scantlings of individual tanks are based, shall be given in the operating manual.

Hydrostatic pressure heads shall be based on approved tank filling by e.g. pumping gravitational effect as well as venting arrangements.

A "load-on-top" system-installed to limit the pressure to h_{pc} can be taken into account.

Pumping pressures can be limited by installing an appropriate alarm and auto-pump cut-off system. In such a situation the pressure head can be taken to be the cut-off pressure head h_{pc} .

Dynamic pressure heads resulting from filling through pipes by pumping are to be included in the design pressure head.

The maximum internal pressure in full tanks is to be taken as the largest of p_1 and p_2 given below:

$$p_1 = \rho g (h_{pc} + h_D) \quad (kN / m^2) \quad (1)$$

where

h_{pc} is the- vertical distance (m) from the action point to the position of max filling height. If no control devices are used, the pressure height should be considered to be top of the air pipe. For tanks adjacent to the sea that are situated below the extreme operational draught (T_E), h_{pc} is not to be taken to be less than T_E

h_D is the dynamic pressure head due to flow through pipes

$$p_2 = \rho g h_s + p_0 \quad (kN / m^2) \quad (2)$$

where

h_s is the vertical distance (m) from the action point to the top of the tank

p_0 is 25 kN/m² in general or the valve opening pressure when it exceeds the general value

For in-deck tanks of topside structures, without opening valves, the pressure should be taken to be p_1 .

Situations where the planned pressure head is exceeded should be considered as a possible ALS condition.

For external plates facing the sea the relevant condition of external pressure should be considered. For internal plates the maximum pressure on each side should normally not be assumed to occur simultaneously.

The tank pressures given in this section refer to static pressures only. When hydrostatic pressure is combined with hydrodynamic pressure caused by the motion of the installation, the pressure p_1 shall be used with the dynamic pressure (h_D) due to flow resistance in the pipe neglected.

5.4.2 Ballast

Tanks, pipes, etc. shall be designed to resist local pressures as described in 5.4.1.

The structure should be designed to resist the maximum uneven distribution of fluid consumables and ballast in tanks that may occur during fabrication, installation and operation.

Pressure actions that may occur during emptying of water or oil filled structural parts for condition monitoring; maintenance or repair shall be evaluated.

5.5 Variable actions in temporary phases

Lifting actions are imposed on the installation or parts thereof during fabrication and installation phases. Such actions should be determined with due consideration of weight and weight growth, shift of centre of gravity, dynamic effects as well as fabrication and sling tolerances.

Guidance on actions and action effects during marine operations may be found in NORSOK J-003.

6 Environmental actions

6.1 General

6.1.1 Environmental conditions

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. Existing information does not suggest significant corrections due to long-term climatological changes, such as the greenhouse effect, which are neglected. Data for the joint occurrence of e.g. wave, wind and current conditions should be aimed at.

According to the regulations, the environmental actions shall be determined with stipulated probabilities of exceedance. Characteristic actions for the design of structures in the in-place condition are defined by annual exceedance probabilities of 10^{-2} and 10^{-4} . The statistical analysis of measured data or simulated data should make use of different statistical methods to evaluate the sensitivity of the result. The validation of distributions with respect to data should be tested by means of recognised methods.

The analysis of the data shall be based on the longest possible time period for the relevant area. In the case of short time series the statistical uncertainty should be accounted for when determining design values. Hind casting can be used to extend measured time series, or to interpolate to places where measured data have not been collected. In this case the hind casting model shall be calibrated against measured data, to ensure that the hind cast statistics results comply with measured data available.

Response in irregular waves should be estimated with due account of the inherent statistical variation. Different realizations of the most critical sea states should, therefore, be used in numerical simulations as well as experimental prediction of the extreme response under such conditions.

6.1.2 Determination of characteristic actions

Actions shall be determined by analysis. When theoretical predictions are subjected to significant uncertainties, theoretical calculations shall be supported by model tests or observations of existing structures or by a combination of such tests and observations.

Hydrodynamic model tests should be carried out to

- confirm that no important hydrodynamic action has been overlooked (for new types of installations, environmental conditions, adjacent structure),
- support theoretical calculations when available analytical methods are susceptible to large uncertainties,
- verify theoretical methods on a general basis.

Wind tunnel tests should be carried out when

- wind actions are significant for overall stability, motions or structural response,
- when available theoretical methods are susceptible to large uncertainties (e.g. due to new type of installations or adjacent installation affects the relevant installation) in order to support or replace theoretical calculations completely,
- there is a danger of dynamic instability.

When the wind conditions are determined for helicopter decks and structures with large motion, wind model tests should, as a rule, be carried out. The Norwegian Maritime Directorate's regulations of 4 September 1987 concerning construction of mobile installations may be regarded as recognised standard for wind tunnel tests.

Model tests may also be relevant to investigate combined wave- and wind effects.

Theoretical models for calculation of actions from icebergs or drift ice should be checked against model tests or full-scale measurements.

Proof tests of the structure may be necessary to confirm assumptions made in the design. Hence, inclining tests of buoyant structures should be carried out to demonstrate the location of the centre of gravity.

In model tests it is important that the model has sufficient similarity to the actual installation and that the test set-up and registration system provide a basis for reliable, repeatable interpretation, see 10.2.7.

Full-scale measurements may be used to update the response prediction of the relevant structure and to validate the response analysis for future analysis. Such measurements may be applied to reduce uncertainties associated with actions and action effects which are difficult to simulate in model scale. However, reliable full-scale measurements require sufficient instrumentation and logging of environmental conditions and responses, see 10.2.8.

6.2 Hydrodynamic actions

6.2.1 Wave data

If wave actions are of major importance for the design, and wave observations at the given or adjacent locations are limited, further wave data should be recorded according to the requirements of NORSOK N-002 concerning environmental data. If limited measurements have been carried out in the area in question, appropriately conservative values should be selected, e.g. by using those given in Figure 1, in order to deal with this uncertainty. Measured data may be replaced by hind cast predictions as indicated in 6.1.

If the directional extremes (sea states or design waves) are to be used for calculating characteristic actions, i.e. for the design of a directionally optimized structure, it should be verified that the actions correspond to the target annual exceedance probabilities. This can be done by doing a full long term analysis where exceedance probabilities are properly considered for the various sea states characterized by H_s , T_p and θ_m , where θ_m is the mean direction of wave propagation.

6.2.2 Wave models

6.2.2.1 General

Waves may be specified by

- a) a long-term description of the sea state climate together with a short-term description of the individual waves in each sea state, see 6.2.2.2,
- b) a short-term design sea state and a description of the individual waves in the sea state, see 6.2.2.3,
- c) a design wave, see 6.2.2.4,

and recognised wave theories.

6.2.2.2 Long-term variation

The variation of waves over a long-term period of several years can be described by a number of stationary sea states each represented by a wave spectrum and a mean direction θ_m (see 6.2.2.3), together with the frequency of occurrence of the main spectral parameters (H_s , T_p) and the mean direction, θ_m . A joint probability density function may be obtained by fitting probabilistic models to a scatter diagram determined by field observations or hind casting. Since this approach would normally imply extrapolation to extreme sea-states beyond the range of observations, recognised models shall be used.

The long-term variation may refer to all year wave conditions or seasonal wave conditions.

The long-term variation of waves can be described in a simplified manner by a number of wave height groups characterised by a wave height, a wave period and the number of waves in the group. This method is not recommended if dynamic effects are significant.

The long-term distribution of waves may be applied to establish a probability distribution for the extreme or fatigue action effect in question. For this purpose a smooth joint model (probability density function) should be

used to represent the long-term variability of sea states. Observed scatter diagrams should be used with care if a limited sample of sea state observations is available, especially in predicting extreme action effects.

Calculation of fatigue action effects would normally require use of a long-term model, in terms of a scatter diagram or joint probability distribution, to account for the variability in sea states.

Each sea state is described in 6.2.2.3.

The possibility of special wave conditions, due to the effect of e.g. shallow water and strong current on wave, should be considered.

6.2.2.3 Design sea states

Calculation of characteristic action effects may be based on selected short-term states. This may especially be necessary for systems significantly influenced by nonlinear behaviour. For such applications the action effects are obtained from time-domain analyses and/or model tests.

The overall aim of the design storm concept is to estimate actions and action effects corresponding to a prescribed annual exceedance probability, e.g. 10^{-2} or 10^{-4} , without having to carry out a full long-term response analysis. The design storm approach is especially relevant in connection with nonlinear action effects.

An appropriate formulation of the design storm concept is to use combinations of significant wave height and spectral peak period located along a contour line in the H_s and T_p plane. Such contour lines can be established in different ways. The simplest way to establish the 10^{-2} contour line, is first to estimate the 10^{-2} value of H_s together with the conditional mean of T_p . The contour line is then estimated from the joint model of H_s and T_p as the contour of constant probability density going through the above mentioned parameter combination. An example of such a contour line is shown in Figure 2. An estimate of the 10^{-2} action effect is then obtained by determining a proper extreme value for all sea states along the contour line and taking the maximum of these values.

If contour lines are used, the variability of the short term extreme value needs to be artificially accounted for in order to obtain a proper short term extreme value. This may be achieved in alternative ways, e.g. by multiplying the expected maximum action effect with a predetermined factor or by calculating the action effects as a predetermined, high fractile value of the 3 h extreme value distribution. To obtain the action effect corresponding to an annual exceedance probability of 10^{-2} the relevant factor would be 1,1 to 1,3 and the fractile should be 85 % to 95 %. When predicting linear or nearly linear response that corresponds to 10^{-2} probability values, the lowest interval limits apply. For lower annual exceedance probabilities, e.g. 10^{-4} and/or clearly non-linear response problems, the fractile should be 90 % to 95 %, see Kleiven and Haver (2003). This simplified approach for obtaining long term characteristic actions should as far as possible be verified by a long-term analysis.

A more consistent method to determine the contour-lines is to apply the inverse FORM approach (see Winterstein et al. 1993).

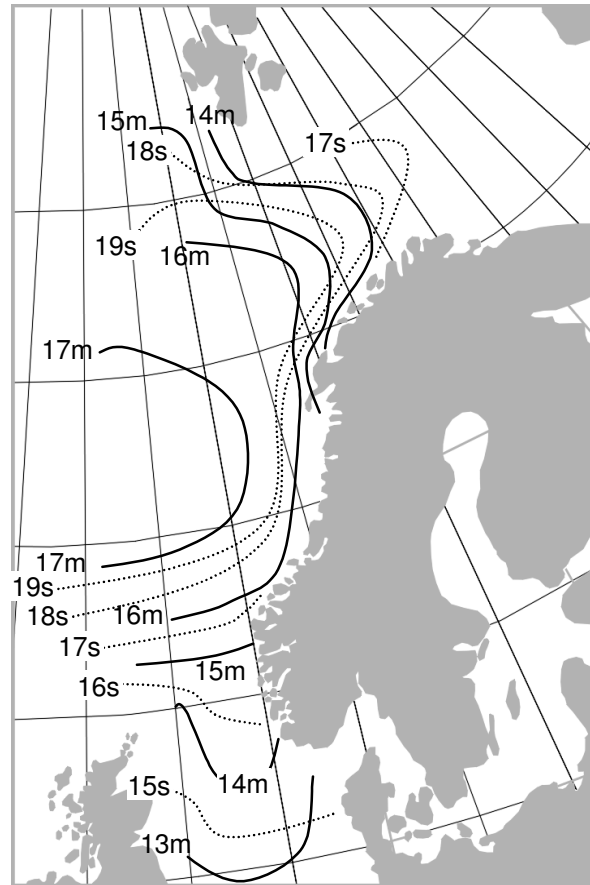


Figure 1 - Significant wave height H_s and related maximum peak period T_p with annual probability of exceedance of 10^{-2} for sea-states of 3 h duration. ISO-curves for wave heights are indicated with solid lines while wave period lines are dotted

Wave spectra should represent wind-induced waves and swell, if relevant. Sea-states comprising unidirectional wind-waves and swell should be represented by recognized double-peaked spectra, e.g. the spectrum proposed by Torsethaugen (2004). The JONSWAP spectrum may be used to describe wind-induced extreme sea states. If wind-waves and swell with different mean directions are critical, due account of such conditions shall be made.

The short-crestedness of wind-induced waves may be described by a spreading function:

$$D(\theta - \theta_m) = C \cos^n(\theta - \theta_m) \quad \text{for } -\frac{\pi}{2} \leq \theta - \theta_m \leq \frac{\pi}{2} \quad (3a)$$

where θ_m is the mean wave direction. In absence of more detailed documentation, the exponent n is taken to be the most unfavourable value between 2 and 10. Swell should be considered to be long-crested.

$$C = \frac{\Gamma(1+n/2)}{\sqrt{\pi} \Gamma(1/2+n/2)} \quad (3b)$$

where Γ is the Gamma function.

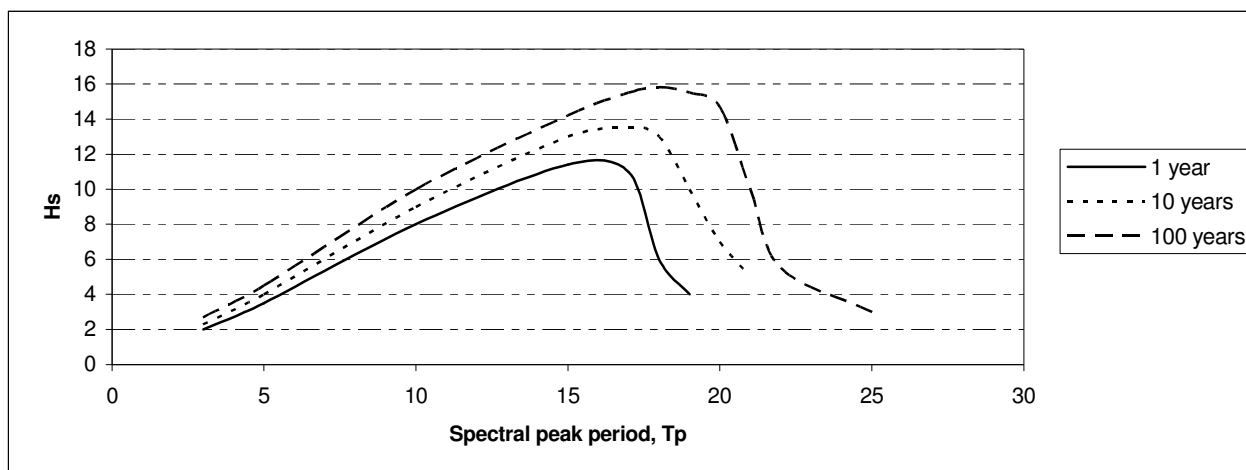


Figure 2 - Example contour line

When calculating the change in the soil's resistance during cyclic actions, the wave conditions shall be described by means of the sea state over an extensive period of time, taking into account the build-up as well as the tail-off phase of the storm. Unless more accurate data are available, the storm development indicated in Figure 3 may be used.

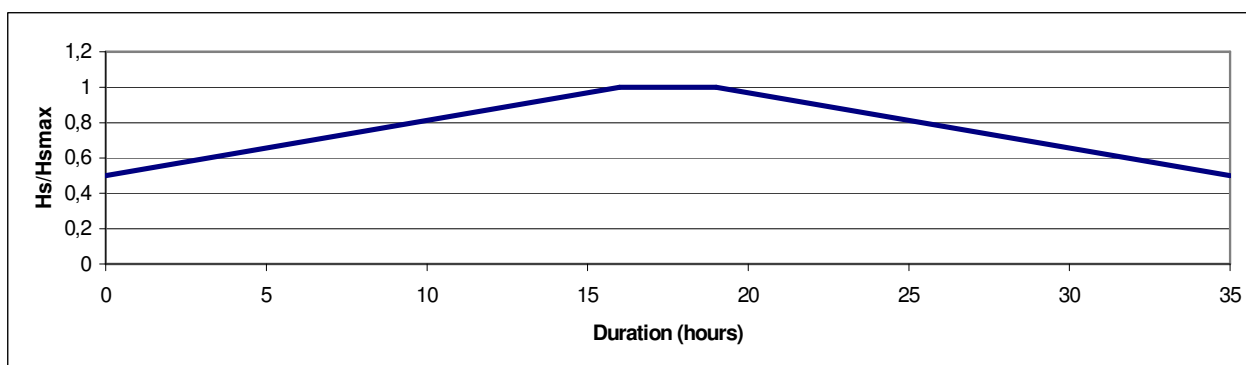


Figure 3 - Storm development for evaluation of the degradation of soil's resistance during cyclic action. The peak level is obtained for a sea-state with duration of 3 h

The random waves in a short-term sea state are normally described as a superposition of regular waves, using linear wave theory. The linear theory should be modified to describe the kinematics in the splash zone e.g. for slender structures, or for situations where steep waves are of particular importance e.g. in case of finite water depth. Commonly used heuristic models are for instance a vertical extrapolation above mean water level, or Wheeler stretching as described e.g. in ISO/DIS 19902. It is noted that the commonly used Wheeler stretching is non-conservative and needs to be modified for steep waves. The heuristic models are limited to symmetric Gaussian waves, and should be appropriately validated for extreme sea states. Alternatively, a non-Gaussian wave model may be obtained by appropriate transformation of the Gaussian process or by applying a second or higher order wave theory for irregular waves. Such theories may be required to model nonlinear high and low frequency actions in irregular waves, see 6.2.6.

In cases where the action effect is influenced by dynamic effects or non-linear drag effects, linear theory shall be modified to improve the description of the wave kinematics and actions in the splash zone. An appropriate way to obtain the kinematics for such cases may be to use second order wave theory to determine the surface elevation and the Wheeler theory to obtain the wave kinematics.

Guidance on the statistical distribution of wave crest elevation may be found in ISO 19901-1.

6.2.2.4 Design wave

A design wave is specified by the wave height, H , the wave period, T , and direction and may especially be used to determine structural action effects which are not influenced by system dynamics. Different combinations of wave periods, wave heights and directions at the same probability level (e.g. 10^{-2} or 10^{-4}) shall be considered in order to arrive at the most unfavourable values for the different action effects.

Consistent sets of (H, T) for each direction may be obtained as long-term contour curves for (H, T) . Such contour curves for (H, T) may be obtained by using information about the short-term probability density function of (H, T) and the long-term distribution of sea-state parameters (H_s, T_p) .

In practice design wave approaches could be used to obtain structural action effects in certain circumstances as described below, but would not be applicable to motion analysis.

Action effects with e.g. annual exceedance probability of 10^{-2} can be determined in a simplified, conservative manner by the design wave approach for preliminary design of fixed platforms. For fixed platforms which respond to wave actions with negligible dynamic effects, maximum action effects occur for the highest waves. The relevant wave height H_{100} is then taken to be that with the 10^{-2} exceedance probability. H_{100} may be taken to be 1,9 times the significant wave height H_s , corresponding to an annual exceedance probability of 10^{-2} , as obtained from long-term statistics, when the duration of the sea-state is 3 h.

The period T used in conjunction with H_{100} should be varied in the following range:

$$\sqrt{6,5 H_{100}} \leq T \leq \sqrt{11 H_{100}}$$

In detailed design special studies of the design wave in the relevant area should be carried out.

In absence of more detailed documentation, the wave height, H_{10000} with annual exceedance probability 10^{-4} can be taken to be 1,25 times H_{100} , while the period is increased by 5 %, as compared to the period of H_{100} .

Maximum action effects are not always caused by waves with extreme heights, but are rather sensitive to waves of a defined length and extreme steepness. Examples are structural action effects in floating installations with columns and pontoons. A simplified, conservative approach to determine the action effect with annual exceedance probability 10^{-2} in such situations may be based on a wave with critical period (length) causing for instance maximum splitting forces in floating framework platforms. The corresponding design wave height, H_d , should be taken to be

$$H_d = \begin{cases} 0,22 T^2 & ; \text{for } T \leq 6 \\ \frac{T^2}{4,5 + 0,02(T^2 - 36)} & ; \text{for } T > 6 \end{cases} \quad (4)$$

for deepwater waves. H_d does not need to be taken greater than 1,9 times the significant wave height corresponding to an annual exceedance probability of 10^{-2} , or the wave height with an annual exceedance probability of 10^{-2} according to the long-term distribution.

Design wave approaches used in detailed design need to be calibrated based on stochastic analysis, as described in clause 10.

The kinematics of the (extreme) design wave may be modelled by Stokes 2nd or higher order theory for water depths (d) to wave length ratios greater than 0,15. Stream function theory is applicable for more shallow water. Wave asymmetry shall be properly accounted for when applying linear wave theory for extreme regular waves. Engineering approximations of the kinematics with acceptable accuracy for extreme waves can be obtained by vertical extrapolation of particle velocities above mean water level, or Wheeler stretching, see ISO/DIS 19902. Linear theory is relevant for calculating action effects for fatigue analysis.

At very shallow water depths other wave theories shall be used, see Sarpkaya T and Isaacson M.

A set of regular waves may be used to establish a relation between stress (range) and wave height for use in simplified long-term action effects for fatigue analysis, see 6.2.2.2. A representative set of regular waves with steepness equal to 1/20 should be used, considering among others waves with a frequency corresponding to the peaks of the transfer function for relevant action effects.

The design wave method should not be used without careful calibration when determining extreme action effects that are influenced by dynamic effects.

Also, special considerations are required to assess possible abnormal wave conditions at an annual exceedance probability of 10^{-4} .

6.2.3 Current velocities

6.2.3.1 Macroscopic current velocity

The current velocity at the location of the installation shall be established on the basis of previous measurements at the actual and adjacent locations, hind cast predictions of wind-induced current, as well as theoretical models and other information about the tidal and coastal current. If the current velocity is of significant importance to the design and existing current data are scarce and uncertain, current velocity measurements should be carried out at the location in question.

Characteristics values of the current velocity should be determined with due account of the inherent uncertainties.

If sufficient joint data about current and wave conditions are available, the joint distribution of environmental parameters and the corresponding contour curves (or surfaces) for given exceedance probability levels can be established in the same way as mentioned for wave conditions in 6.2.2.3. Otherwise, conservative values, for instance using the combined events indicated in Table 4, should be applied.

In early phases and in exploration drilling where no accurate measured data or documented model studies are available, the tidal current at still water level may be chosen in accordance with Figure 4.

If no exact measured data are available, the wind induced current velocities at still water level may be selected equal to 2 % of the 1 h mean wind velocity at a 10 m elevation, see 6.3.

For exploration drilling installations and in early phases of a development where no accurate measured data or documented model results exist, the current variation with depth may be chosen in accordance with DNV Class Note 30.5.

In addition, the effects of other currents shall be considered in each separate case, including the effects of

- coast and ocean currents,
- local eddy currents,
- currents over steep slopes,
- currents caused by storm surge,
- internal waves.

When calculating erosion, it shall be taken into consideration that the structure may change the local current velocity.

6.2.3.2 Current profile at a structure

The current speed in the vicinity of the platform may be reduced from the specified "free stream" value by blockage. However, reduced actions on risers or conductor arrays due to blockage may cause increased actions on adjacent structural components, e.g. due to increased particle velocities in those areas.

In absence of a detailed evaluation, a blockage factor of 0,9 and 0,85 can be used for jackets with three legs and more than three legs, respectively, and static behaviour. Further details are given in ISO/DIS 19902.

Blockage factors for platforms with significant dynamic behaviour can be applied, if the effects are adequately documented.

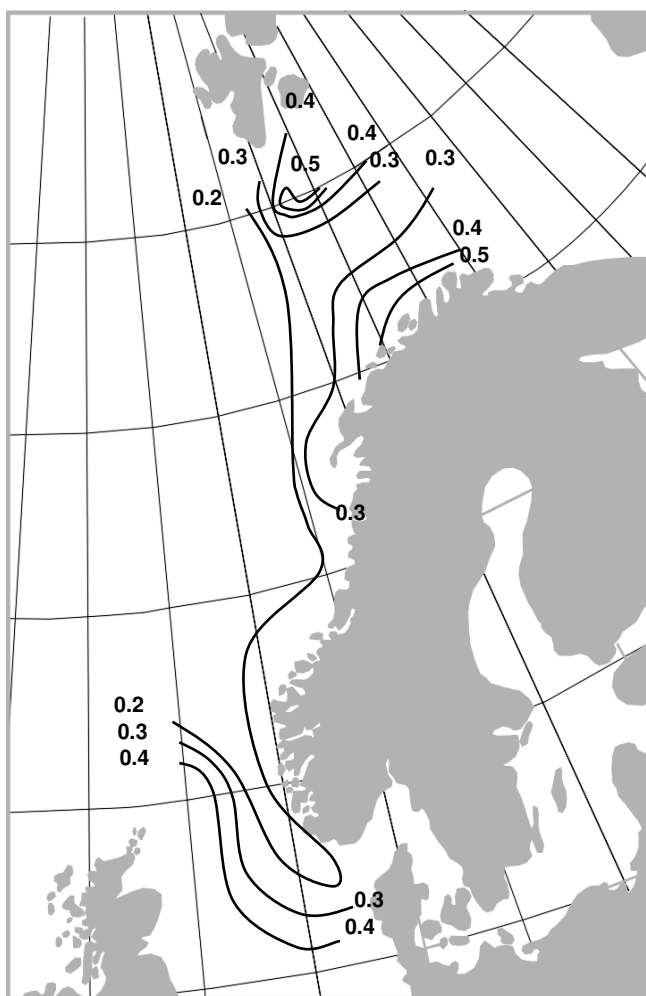


Figure 4 - Maximum 100 year tidal surface current in m/s

6.2.4 Hydrodynamic actions for static analysis

6.2.4.1 Wave and current effect

Waves and current should be considered when calculating hydrodynamic actions. In combination with waves, the current velocity profile should be stretched to the local water surface.

The hydrodynamic action on slender members, i.e. with a wave length to member diameter greater than 5, can be determined by Morison's equation using a particle velocity obtained by vector addition of wave and current induced particle velocities.

For large volume structures the current/wave/body interaction should be considered when deriving resultant actions.

6.2.4.2 Slender tubular structural elements

For structures with small motions, the wave actions can be calculated as follows:

- a) if the KC is less than 2 for a structural element, the actions may be found by means of potential theory:
 - 1) if the ratio between the wave length L and the tubular diameter D is greater than 5, the inertia term in Morison's equation can be used with $C_M = 2,0$;
 - 2) if the ratio between L and D is smaller than 5, the diffraction theory should be used.
- b) if KC is greater than 2, the wave action can be calculated by means of Morison's equation, with C_D and C_M given as functions of the Reynold's number, the KC and relative roughness;

It should be noted that Morison's equation ignores lift forces, slam forces and axial Froude-Krylov forces.

- c) for surface piercing framed structures consisting of tubular slender members (e.g. conventional jackets) extreme hydrodynamic actions on unshielded circular cylinders are calculated by Morison's equation on the basis of
- 1) Stokes 5th order or Stream function wave kinematics and a kinematics factor on the wave particle velocity, which is 0,95 for North Sea conditions. This kinematics factor is introduced in the regular wave approach to account for wave spreading and irregularity in real sea states,
 - 2) drag and inertia coefficients equal to
 - $C_D = 0,65$ and $C_M = 1,6$ for smooth members
 - $C_D = 1,05$ and $C_M = 1,2$ for rough members

These values are applicable for $u_{\max} T_i / D > 30$, where $u_{\max}$ is the maximum horizontal particle velocity at storm mean water level under the wave crest, T_i is the intrinsic wave period and D is the leg diameter at the storm mean water level.

- d) flow conditions with $u_{\max} T_i / D < 30$ in regular waves may arise with slender members in moderate sea-states which are relevant for fatigue analysis;

Fatigue analysis can normally be conducted with no current. The wave kinematics factor and conductor-shielding factor should be taken to be 1,0. C_D and C_M depend on the sea state level, as parameterised by KC . For small waves with KC referred to the mean water level in the range $1,0 < KC < 6$, the hydrodynamic coefficients can be taken to be

$$C_D = 0,65 \text{ and } C_M = 2,0 \text{ (smooth member)}$$

$$C_D = 0,8 \text{ and } C_M = 2,0 \text{ (rough members)}$$

Members are considered smooth at the installation stage. During operation members 2 m above mean water level are considered smooth, see 6.6.1.

For KC in the range of 6 to 30 the C_D and C_M should be determined by special considerations.

- e) the action effects in slender structures which are exposed to significant wave induced dynamics, should be assessed by means of time domain simulations;

It is recommended that the surface elevation process is modelled as a second order process with its corresponding second order kinematics. No kinematic reduction factor is to be used. The kinematics may as an alternative be estimated by approximating the second order surface process by Fourier series and calculate the kinematics for each component utilizing Wheeler stretching. In both these models, the hydrodynamic coefficients in item c) should be applied. If the actions, their effects and responses from the time domain simulations are to be used in an absolute sense, the adequacy of the applied kinematics and hydrodynamic coefficients should be verified.

If the sea surface is modeled as a Gaussian process, the hydrodynamic coefficients should be calibrated to give a reasonable quasistatic load level in view of the purpose of the simulation. If extremes are under consideration, a reasonable load level should be obtained for the largest waves of the most severe sea state, while for fatigue a reasonable level for the most important fatigue accumulating waves, should be ensured. As a first approach one may for a drag dominated structure use C_D equal to 1,15 and C_M according to item c).

For fatigue assessment a Gaussian sea surface would normally be sufficiently accurate. Regarding extreme value considerations, the adequacy of applying a Gaussian surface for estimation of dynamic amplification should be assessed on a case by case basis.

- f) wave actions on conductors/risers may contribute to the global actions on structures. If conductors/risers are closely spaced the actions on them may be modified as compared to actions on individual components, due to hydrodynamic shielding. Guidance on the shielding factor for drag actions when the fluid flows in parallel with the main axes of a rectangular array of cylinders may be found in ISO/DIS 19902. When the angle between the wave or current direction and the direction of the rows of cylinders is between $22,5^\circ$ and $67,5^\circ$, the shielding factor in ISO/DIS 19902 can be used when the spacing is determined as the average for the two directions. A possible increase in the added mass (and inertia actions) for closely spaced cylinders should be accounted for.

Shielding reduction factors should not be applied for platforms without documentation.

6.2.4.3 Large volume structures

Actions on fixed large volume structures should be calculated on the basis of diffraction theory. The potential is expressed as the linear superposition of

- the incident wave system,
- the scattered wave system.

For simple geometrical shapes analytical solutions of the linear diffraction problem may be used. For slender structures, strip theory may be applied.

For general structures consisting of several large volume components, boundary-element or finite fluid elements should be used.

The results from boundary element methods should be carefully checked for surface-piercing bodies to ensure that irregular frequencies are avoided. Convergence of the solution should be demonstrated.

Estimates of actions for novel structural shapes need to be checked by model tests.

6.2.4.4 Hybrid structures

Wave diffraction solutions do not include viscous actions. When body members are relatively slender or have sharp edges, viscous effects may be important and viscous effects should be added to the diffraction forces determined.

Wave actions on structures composed of large volume parts and slender members may be computed by a combination of wave diffraction theory and Morison's equation. Parts of the structure may be modelled both by boundary elements to represent the potential hydrodynamic actions and beams to represent the viscous drag actions. The modifications of velocities and accelerations as well as surface elevation due to the large volume parts should, however, be accounted for when using Morison's equation.

If properly calibrated or validated, simplified conservative methods may be used to calculate actions.

6.2.4.5 Effect of adjacent structure

If a structure is located adjacent another structure and one or both of them are large volume structures, there may be an interaction between the actions on the two structures.

A diffraction analysis is then required for the total problem. Model tests need to be considered if the analysis involves uncertainties, e.g. associated with extreme actions situations.

6.2.5 Hydrodynamic actions for dynamic analysis

6.2.5.1 General

Actions on structures with significant motions involve excitation actions as well as inertia, damping and restoring actions as formulated by the dynamic equation of motion.

6.2.5.2 Slender structures

When the motion amplitude is greater than the diameter, the relative motion between the structure and water should be taken into consideration when calculating wave and current actions by means of Morison's equation. The dynamic equilibrium equation for a structure may then formally be expressed as

$$(\mathbf{M} + \mathbf{M}_A) \ddot{\mathbf{X}} + \mathbf{C} \dot{\mathbf{X}} + \mathbf{K} \mathbf{X} = \mathbf{F} \quad (5a)$$

where $\mathbf{M}$, $\mathbf{M}_A$, $\mathbf{C}$ and $\mathbf{K}$ are structural mass, added mass, structural damping and structural stiffness, respectively.

NOTE It is noted that Eq.(5a) needs to be modified if stochastic time domain analysis is carried out for systems with frequency dependent dynamic properties.

The external action vector $\mathbf{F}$ is obtained by integrating the following action per unit length over each member of the structure

$$d\mathbf{F}_N = C_D \frac{1}{2} \rho D |\mathbf{U}_{RN}| \mathbf{U}_{RN} + C_M \rho \pi \frac{D^2}{4} \dot{\mathbf{U}}_N \quad (5b)$$

and decomposing the action in its Cartesian components. $\mathbf{U}_{RN}$ and $\dot{\mathbf{U}}_N$ are the relative velocity, and acceleration normal to the member, respectively.

Caution should be exercised when using this approach to determine wave excitation, added mass and damping actions at intersections between large diameter members.

C_D and C_M are the same hydrodynamic coefficients, as used for fixed structures.

$\mathbf{X}$, $\dot{\mathbf{X}}$ and $\ddot{\mathbf{X}}$ are the displacements, velocity and acceleration of the structure.

The formulation of actions in terms of relative velocity is uncertain for installations with small motion. The method may in such cases give too high damping. Hydrodynamic damping should therefore be taken into consideration through an equivalent viscous damping, and the particle velocity should be used instead of relative motion.

When using the relative velocity formulation for structures with large motion, actions should be obtained for zero as well as maximum current velocity, and with a low and high value of C_D , depending upon whether the drag actions primarily induce damping or excitation.

The action coefficients may be introduced for the action effect, and not on the actions, in such calculations.

6.2.5.3 Large volume structures

For large volume structures with significant motion a radiation wave system generated by the moving structure should be added to the incident and scattered wave system.

The radiation wave system is determined by diffraction analysis (see 6.2.4.3) and gives rise to added mass and wave damping actions.

In addition, viscous actions on the hull, a possible mooring system or thrusters and risers should be considered.

Viscous actions due to wave- and low-frequency motions may be calculated separately by a relative velocity formulation and superimposed or calculated simultaneously, using appropriate phase for the actions, as well as hydrodynamic coefficients, as explained in clause 10.

6.2.6 Higher order, nonlinear wave actions

Higher order terms in the potential theory or finite wave height kinematics used in conjunction with Morison's equation, cause mean and time-variant sum and difference frequency action in irregular waves which may cause significant response if resonance occurs. Similar effects may be present for (large volume) structures that require a more elaborate method for determining actions.

The higher order actions should be determined by a consistent theory and validated against model tests.

Difference (low) frequency actions associated with wave-body nonlinear interaction, may be important for global motions and positioning systems when the action synchronise with fundamental periods of vibration of the system or its parts. Due to the uncertainty associated with calculations of slowly varying drift motions (Herfjord and Nielsen, 1991), model tests would be required to reduce the uncertainty, when the response is significant.

Sum (high) frequency actions (causing springing) may be important for the response of restrained modes of tension-leg platforms, ships and jack-ups, of importance especially for the fatigue limit state.

Steep, high waves encountering structural components extending above the still water level may cause nonlinear transient actions. Structural responses to these actions may be dynamically amplified and cause

increased extreme response (ringing). Such transient nonlinear actions may be important for structures consisting of large diameter shafts and having a natural period under 8 s. However, the physical mechanisms causing ringing are not known, although it is generally accepted that they can be described by potential theory. One method that seems to predict ringing actions reasonably well is presented by Krokstad et al. (1996). Currently available analysis methods are generally only amenable to screening analysis of the ringing problem. The phenomenon is today best quantified by model tests. It is then generally difficult to distinguish impact/slamming from higher order inertia (ringing) effects.

With regard to structures that have been optimised with respect to minimal linear wave actions, experience has shown that non-linear effects may be dominant. Motion analyses for new types of structures where the results cannot be checked against previous experience, should be checked against model tests.

6.2.7 Wave slamming and run-up effect

Slamming of horizontal or near horizontal surfaces that enter the water or are hit by a wave should be assessed. The corresponding action should be investigated by momentum considerations of potential actions together with account of viscous and buoyancy effects. The effect of elasticity of the structure should be considered.

Horizontal members in the splash zone are susceptible to actions caused by wave slamming when the member is being submerged. The slamming action may be estimated by the "dragterm" of Morison's equations based on the relative velocity of the member to the water particle and with an appropriate drag (slamming) coefficient. For smooth cylindrical members the coefficient shall not be taken less than 3,0. For a flat plate a coefficient of 6,0 may be used to estimate the average pressure over the plate. An even larger coefficient is applicable for more local pressures. When the slamming force is determined by a "dragterm" due consideration of a consistent choice of coefficient and area of contact should be made.

The method established by Ridley (1982) can be used to estimate splash-zone wave actions on inclined slender tubular members.

Since slamming is an impulse action, a dynamic amplification factor of at least 2,0 should be used, unless an analysis, which considers the duration of the impulse action and natural frequency of the structure, demonstrates that a lesser value is appropriate.

Breaking waves causing shock pressure in vertical surfaces, should be considered. Such actions are determined by the procedure given by DNV Classification Note 30.5.

Wave actions on decks consisting of plated and tubular structural components comprise inertia, slamming, drag and buoyancy actions.

A simplified approach for determining horizontal wave actions on decks of fixed platforms is given by API RP 2A-LRFD, clause 17 (ISO/DIS 19902). An alternative method that also includes vertical actions on the deck is outlined and compared with model test results by Kaplan et al (1995).

Appropriate consideration shall be given to the possibility of slamming events occurring to underwater structural arrangements as a result of lack of depth of submersion.

If dynamic effects are of importance to the action effects, both the water entry and exit phase should be duly modelled.

The possible effect of the up-welling (run-up) caused by a large, steep wave passing a large diameter vertical cylinder or columns with flat areas should be considered in designing the lower deck structure.

While the principal effect of slamming and up-welling is local, possible global effects should also be assessed.

Slamming on a ship may occur in the forward bottom, bow flare, accommodation structure in the fore ship as well as exposed parts of the aft structure and turret. Slamming effects depend upon the following features of the vessel:

- draught;
- hull geometry;
- superstructure arrangement;
- vessel speed;

- heading.

Slamming actions may contribute local effects as well as enhancement of hull girder bending moment and shear force.

In lack of more accurate information bottom and bow flare slamming may be determined according to the DNV Rules for Classification of Ships, appropriately modified to account for the relevant operational pattern of production ships.

6.2.8 Green water effect

Green water is the overtopping by water of the part of a structure facing the seas in severe wave condition. For a ship the phenomenon depends on bow shape, design free board, flare, breakwaters and other protective structures, as well as drainage arrangements, and weather vaning procedures. Green water may enter a ship in the bow as well as mid-ship and aft. Significant amounts of green water will have influence on the design of the vessel deck, accommodation superstructure, equipment and layout.

Only rough estimates of green water can be made by analytical methods and model tests should be carried out to estimate the green water level and (dynamic) pressures, if more accurate estimates are required.

In lack of more accurate information deck structural members could be designed based on the actions given in DNV-OS-C102, appropriately increased to account for the relevant operational pattern of the ship.

Model tests should be carried out to determine pressure levels in the deck area in wave conditions with a peak period in the range of 70 % to 100 % of the vessel pitch period.

The effect of wave slamming, run-up or green water actions should be appropriately combined with the other wave action effects.

6.2.9 Wave enhancement

The wave enhancement and modification of the kinematics caused by the structure may be estimated by linear radiation and diffraction theory. The fact that the wave crest is higher than given by linear theory shall be accounted for.

Model tests should be performed if the theoretical methods cannot predict the effect of substructure with sufficient confidence due to nonlinearities related to steep waves, and wave-current interaction. This situation may arise in connection with caissons of gravity structures and shallow pontoons of floating platforms, strong interaction between large columns, non-vertical sides near the water plane, and other features.

Wave motion in moon-pool or centre well of ship and caisson vessels (spar buoys) should be particularly accounted for.

6.2.10 Sloshing actions in tanks

Motion induced dynamic pressure effects in partially filled tanks, especially with large horizontal dimensions, should be considered. Sloshing represents a dynamic magnification of internal pressure effects beyond the static pressure. Sloshing occurs if the natural periods of the fluid in tanks and of the vessel motions are close to each other. Possible impact effects should be accounted for in this connection.

Sloshing effects depend upon tank dimensions and filling levels, structural arrangement inside the tank and vessel motion characteristics.

Simplified formulas given in DNV-OS-C102 or direct calculation methods may be applied to determine sloshing actions.

6.2.11 Flow-induced vibrations

Current induced vibrations shall be taken into account, e.g. in terms of

- a) vortex-induced vibration,
- b) instability caused by varying orientation of the structure in relation to the wave and current direction (galloping),
- c) back wash effects from units in the vicinity.

Actions by vortex shedding may be of significant importance in the design of slender structures that may respond in resonance modes to this cyclic action, especially when the damping is small. Critical flow velocities for vortex shedding for current and wave excitation are given in DNV Class Note 30.5.

The excitation may be characterised by the motion amplitude and/or the forces on the member. Vortex shedding of larger parts of the structure due to current or wave excitation should also be considered.

VIV will increase the mean in-line drag coefficient. Unless more accurate data are available, the drag coefficient valid for a stationary cylinder should be multiplied with a factor which depends upon the ratio of the transverse motion and the member diameter, in order to include the VIV induced drag amplification.

Vortex shedding may especially contribute to fatigue. The cumulative effects of resonant actions during construction, transportation and operation shall be included in the calculations.

Vortex shedding may be reduced by introducing devices to prevent or reduce the vortex intensity, or by changing the vibration properties, e.g. natural period and damping of the structure.

Further guidance of vortex-induced vibration may be found in DNV Classification Note 30.5.

6.3 Wind actions

6.3.1 Wind data

The wind velocity at the location of the installation shall be established on the basis of previous measurements at the actual and adjacent locations, hind cast predictions as well as theoretical models and other meteorological information. If the wind velocity is of significant importance to the design and existing wind data are scarce and uncertain, wind velocity measurements should be carried out at the location in question.

Characteristic values of the wind velocity should be determined with due account of the inherent uncertainties.

6.3.2 Description of wind

For a short term condition the wind may be described by means of an average wind velocity and a superimposed fluctuating wind gust with a mean value equal to zero, as well as a mean direction.

Unless a more detailed assessment is made, the average wind velocity at 10 m above sea level the characteristic value with an annual probability of exceedance of 10^{-2} can be chosen as 41 m/s (10 min average) or 38 m/s (1 h average) for the whole continental shelf. The characteristic value with an annual probability of exceedance of 10^{-4} can be chosen as 48 m/s (10 min average) or 44 m/s (1 h average).

The characteristic wind velocity $u(z,t)(\text{ms}^{-1})$ at a height $z(\text{m})$ above sea level and corresponding averaging time period t less than or equal to $t_0 = 3600 \text{ s}$ may be calculated as

$$u(z,t) = U(z) (1 - 0,41 I_u(z) \ln(t/t_0)) \quad (6)$$

where the 1 h mean wind speed $U(z)(\text{ms}^{-1})$ is given by

$$U(z) = U_0 \left[1 + C \ln\left(\frac{z}{10}\right) \right] \quad (7)$$

$$C = 5,73 \cdot 10^{-2} (1 + 0,15 U_0)^{0,5}$$

and where the turbulence intensity factor $I_u(z)$ is given by

$$I_u(z) = 0,06 [1 + 0,043 U_0] \left(\frac{z}{10}\right)^{-0,22} \quad (8)$$

where $U_0 (\text{ms}^{-1})$ is the 1 h mean wind speed at 10 m

For structures and structural components with significant dynamic response under wind fluctuations a wind spectrum shall be used to describe the longitudinal wind speed fluctuations.

For situations where the low-frequency excitation is of importance, the following one sided energy density spectrum of the longitudinal velocity fluctuations at a particular point in space is recommended, see Andersen and Løvseth (1992):

$$S(f) = 320 \frac{\left(\frac{U_0}{10}\right)^2 \left(\frac{z}{10}\right)^{0,45}}{(1 + \tilde{f}^n)^{\frac{5}{3n}}} \quad (9)$$

where $n = 0,468$

$$\tilde{f} = 172 f \left(\frac{z}{10}\right)^{2/3} \left(\frac{U_0}{10}\right)^{-0,75} \quad (10)$$

where

$S(f)$ ($\text{m}^2 \text{s}^{-2} / \text{Hz}$) is the spectral density at frequency f (Hz)

z (m) is the height above sea level

U_0 (ms^{-1}) is the 1 h mean wind speed at 10 m above sea level.

The Harris wind spectrum may be considered when action effects in structures such as flare towers, which are sensitive to the high frequency excitation are to be calculated.

Wind gusts have three-dimensional spatial scales related to their duration, e.g. 3 s gusts are coherent over shorter distances and therefore affect smaller structural elements than 15 s gust. Wind actions on different substructures are normally specified by a given averaging time for the wind speed and assuming full coherence over the entire substructure. Specific information about averaging time is given in 6.3.3 for static and in 6.3.4 for dynamic analysis.

For dynamic analysis of some structures, it may be advantageous to account for the less-than-full coherence at higher frequencies. In this connection coherence spectra given by Andersen and Løvseth (1992) can be used.

6.3.3 Mean wind actions

Structures or structural components that are not sensitive to wind gusts may be calculated by considering the wind action as static.

In the case of structures or structural parts where the maximum dimension is less than approximately 50 m, 3 s wind gusts may be used when calculating static wind actions.

In the case of structures or structural parts where the maximum length is greater than 50 m, the mean period for wind may be increased to 15 s.

When design actions due to wind need to be combined with extreme actions due to waves and current, the mean wind speed over a 1 min period can be used. A longer averaging period may be used if properly documented.

The mean wind action, F , on a structural member or surface, acting normal to the member axes or surface, should be calculated by:

$$F = \frac{1}{2} \rho C_s A U_m^2 \sin \alpha \quad (11)$$

where

ρ is the mass density of air

C_s is the shape coefficient

A is the area of the member or surface area normal to the direction of the force

U_m is the wind speed

α is the angle between the direction of the wind and the axis of the exposed member or surface

For smooth circular tubular structures, the following shape coefficients may be used:

$C_S = 0,65$ for Reynold's number $> 5 \times 10^5$
 $C_S = 1,2$ for Reynold's number $< 5 \times 10^5$

In the case of tubular structures covered with ice, $C_S = 1,2$ should be used for all Reynold's numbers.

Further information about shape coefficients, C_S may be found in ENV 1991-2-4 and DNV Classification Note 30.5.

A simplified method for determining wind forces on ships are given by OCIMF (1994).

Wind pressures and resultant actions shall be determined from wind tunnel tests on representative model when wind actions are crucial.

6.3.4 Fluctuating wind actions

Structures that are sensitive to wind gusts shall be calculated by considering the wind action as a dynamic action. Examples of such structures are high towers, flare booms, tension-leg installations, compliant installations and catenary anchored installations.

The mean wind velocity is based on a 1 h period.

The total wind speed is the sum of a mean velocity and a fluctuating component $u(t)$. The wind force on a structure with a velocity which is negligible compared to the wind velocity, can be calculated by:

$$F = \frac{1}{2} \rho C_S A [U_m + u(t)]^2 \approx \frac{1}{2} \rho C_S A [U_m^2 + 2U_m u(t)] \quad (12)$$

This means that the fluctuating wind force is linear in the fluctuating velocity.

Low-frequency wind forces are normally determined from the wind energy spectrum, see Eq.(9) and Eq.(10).

For structures of large size, the spatial variation of the fluctuating wind could be incorporated in the analysis by accounting for the coherence.

6.3.5 Wind-induced vibrations

Consideration of actions from vortex shedding on individual elements due to wind may be based on ENV 1991-2-4. Guidelines on how these standards may be conservatively applied offshore, are provided by Oppen (1996). Alternative, well documented methods can be applied. Vortex induced vibrations of frames should also be considered. The material-and structural damping of individual elements in welded steel structures should not be set higher than 0,15 % of critical damping when vortex induced vibrations are considered.

6.4 Snow and ice actions

6.4.1 Snow actions

The snow actions given in NS 3491-3 for the relevant coastal municipality may be used as extreme snow action close to the shore. For other areas where more accurate meteorological observations have not been performed, characteristic snow action may be set equal to 0,5 kPa for the entire Norwegian continental shelf.

The shape factors given in NS 3491-3 may be used.

6.4.2 Ice actions

6.4.2.1 Accumulated ice

In absence of a more detailed assessment values for thickness of accumulated ice caused by sea spray or precipitation may be selected as indicated in Table 2. These may be regarded as two independent action cases.

When calculating wave, current and wind actions, increases in dimensions and changes in the shape and surface roughness of the structure as a result of accumulated

- a) ice from sea spray which covers the whole circumference of the element,

- b) ice from rain covers all surfaces facing upwards or against the wind. For tubular structures it may be assumed that ice covers half the circumference.

Uneven distribution of ice shall be considered for buoyancy stabilised structures.

Table 2 – Ice actions with annual probability of exceedance 10^{-2}

Height above Sea level mm	ACTION CASE 1			ACTION CASE 2	
	Ice caused by sea-spray			Ice caused by rain / snow	
	56° N to 68° N mm	North of 68° N mm	Density kg/m ³	Thickness mm	Density kg/m ³
5 to 10	80	150	850	10	900
10 to 25	Linear reduction from 80 to 0	Linear reduction from 150 to 0	Linear reduction from 850 to 500	10	900
Above 25	0	0	-	10	900

6.4.2.2 Frost burst

The effects of ballast water, firewater etc. which may freeze shall be taken into account.

6.4.2.3 Sea ice and icebergs

Actions from sea ice and icebergs shall be taken into account when structures are located in areas near shore, in Skagerak, in the northern and western parts of the Norwegian Sea and in parts of the Barents Sea.

Before activities are commenced in such areas, the following information concerning ice conditions shall be collected:

- the possibility of icebergs and sea ice;
- type of sea ice (first year ice; ice several years old) and characteristic features (ridges, large interconnected floes and individual floes, distribution, etc.);
- sea ice thickness;
- size and shape of icebergs;
- velocity and direction of drifting sea ice and icebergs;
- mechanical properties of the ice.

There shall then be an emergency preparedness system established which will ensure safety in the event of ice. Solutions based on relocation of the installation in the event that effects of sea ice or icebergs may become unacceptable, may be chosen. In such cases the emergency preparedness shall be reliable, and shall be planned in relation to the time required to relocate the installation.

The occurrence of first year ice with annual probability of exceedance of 10^{-2} in the Barents Sea is shown in Figure 5. For planning of operations, the monthly extreme ice limit with annual probability of exceedance of 10^{-2} may be used. As the charts showing ice occurrence are made on the basis of satellite data, and as the concentration must be 10 % to 20 % in order to be detected, the fact that ice with lower concentrations may occur outside the defined limits shall be taken into account. Monthly values for the extreme ice limit with an annual probability of exceedance of 10^{-2} may be found in Vefsnmo et al (1990). These values may be used in evaluations during an early phase.

To calculate the actions caused by ice, values for thickness and size of ice floes that are representative to the area shall be selected. To describe the mechanical properties, the same properties for the sea ice as for ice in other arctic areas may be assumed. API RP 2N may then be used.

Regions where collision between icebergs and an installation can occur with an annual probability of exceedance of 10^{-2} and 10^{-4} in the Barents Sea are shown in Figure 6. Icebergs in considerable numbers have been observed on the coast off East Finmark in 1881 and in 1929.

When designing against icebergs the same principles as stipulated in 8.3.2 of the guidelines may be used.



Figure 5 - Limits of sea ice in the Barents Sea with annual probability of exceedance of 10^{-2} (solid line) and 10^{-4} (dotted line)



Figure 6 - Limit for collision with icebergs with a probability of exceedance of 10^{-2} (solid line) and 10^{-4} (dotted line)

6.5 Earthquake actions

6.5.1 Basis for seismic assessment

Earthquake actions should be determined on the basis of the relevant tectonic conditions, and the historical seismological data. Measured time histories of earthquakes in the relevant area, or other areas with similar tectonic conditions may be adopted.

Earthquake motions at the location may be described by means of response spectra or standardised time histories with the peak ground acceleration to characterise the maximum motion.

The earthquake motion can be described by two orthogonally horizontal oscillatory motions and one vertical motion acting simultaneously. These motion components are assumed to be statistically independent. One of the horizontal excitations should be parallel to a main structural axis, with the major component directed to obtain the maximum value for the response quantity considered. Unless more accurate calculations are performed, the orthogonal horizontal component may be set equal to 2/3 of the major component and the vertical component equal to 2/3 of the major component, referred to bedrock. The earthquake may be scaled with the given factors.

In absence of more detailed, site specific assessments, the peak ground acceleration at annual exceedance probabilities of 10^{-2} and 10^{-4} given in seismic zonation maps in NFR/NORSAR (1998) can be applied.

When determining earthquake actions on the structure, interaction between the soil, the structure and the surrounding water should be taken into consideration.

When time histories are used, the load effect should be calculated for at least three sets of time histories. The mean value of the maximum values of the calculated action effects from the time history analyses may be taken as basis for design. The time series shall be selected in such a way that they are representative of earthquakes on the Norwegian continental shelf at the given probability of exceedance.

6.5.2 Seismic design of structure and foundation

Earthquake design includes ULS (strength) check of components based on earthquakes with an annual probability of occurrence of 10^{-2} and appropriate action and material factors; as well as an ALS check of the overall structure to prevent its collapse during earthquakes with an annual probability of exceedance of 10^{-4} with appropriate action and material factors, given in NORSOK N-001..

Normally the ALS requirement will be governing, implying that earthquakes with an annual probability of exceedance of 10^{-2} can be disregarded.

The assessment of earthquake effects should be carried out with a refinement of analysis methodology that is consistent with the importance of such effects. The assessment of soil-structure integrity could therefore be undertaken in the following steps:

- 1) Determine appropriate peak ground acceleration given in seismic zonation maps, see e.g. NFR/NORSAR (1998).
- 2) Determine whether further checks will be necessary, by e.g. the procedure given in 6.5.3.

For most parts of most structures on the Norwegian Continental Shelf no further check will be necessary. If necessary, more detailed checks, proceed as follows:

- strength check of soil-structure system under 10^{-2} and 10^{-4} events based on linear elastic response using modal superposition, see 10.3.7.1;
- if strength check is not fulfilled, a ductility check may be carried out, see 10.3.7.2;
- if the ductility check is not fulfilled, a more accurate analysis of the site specific seismic hazards than those given in 6.5.3 may be carried out and used to demonstrate compliance with the requirements;
- if the ductility check is still not fulfilled, modification of the design is necessary.

6.5.3 Response spectra for a single degree of freedom system

Unless more accurate assessments are performed, response spectra in Figure 7 may be used for structural action effects. Spectral displacement, velocity and acceleration (normalised to 10 m/s^2 acceleration at 40 Hz for the horizontal main component) are given as a function of the natural period of the structure including its interaction with the soil for various soil conditions. The spectra may be used together with the accelerations given in seismic zonation maps. If for instance the acceleration is $2,5 \text{ m/s}^2$, the spectrum with an annual probability of exceedance of 10^{-4} is obtained by multiplying the normalised spectrum in Figure 7 with 0,25.

The spectrum is based on the total structural and soil damping of 5 % of critical damping. The spectrum shall be adjusted if the damping differs from this value. The spectrum can be scaled with regard to expected acceleration level at 40 Hz or with regard to expected velocity level at 1 Hz. The scaling factor for the spectrum due to variation of the (percentage) damping between 2 % and 10 %, the following formula may be used:

$$K = \ln(100/\zeta) / \ln(20) \quad (13)$$

where ζ is the percentage of damping.

Before detailed assessment of earthquake actions is undertaken, a conservative estimate of the global (e.g. base shear) force based on a single dynamic mode of response and the response spectrum (see Figure 7) may be used to judge the need for such an analysis.

It should be noted that the amplification factor given for soft soils for the 10^{-4} per year event may be conservative for periods greater than 4 s. On the other hand a site specific soil response study is recommended for very soft soils, since they may be outside the range of data used to establish Figure 7, b), see e.g. NFR/NORSAR (1998).

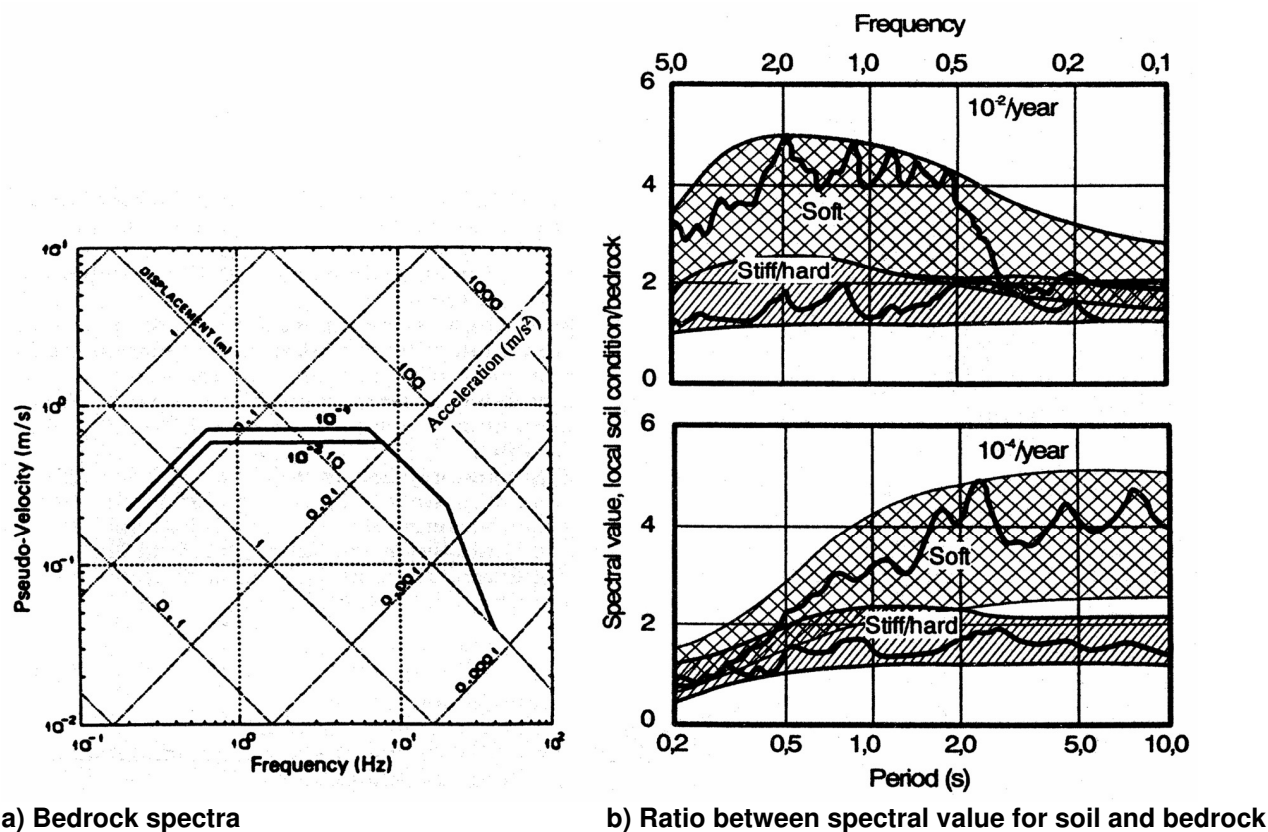


Figure 7 - Response spectra (normalised to 10 m/s^2 acceleration at 40 Hz) with 5% damping. Maximum acceleration, velocity and displacement (Bungum and Selnes, 1988)

6.6 Other issues relating to environmental actions

6.6.1 Marine growth

Marine growth is a common designation for a surface coat on marine structures, caused by plants, animals and bacteria. Marine growth may cause increased hydrodynamic actions, increased weight, increased hydrodynamic additional mass and may influence hydrodynamic instability as a result of vortex shedding and possible corrosion effects.

In the calculation of structural actions, unless more accurate data are available, or if regular cleaning is not planned, thickness referring marine growth to mean water level as indicated in Table 3 may be assumed.

Table 3 – Thickness of marine growth ^a

Water depth m	56° to 59° N mm	59° to 72° N mm
Above + 2	0	0
+2 to - 40	100	60
Under - 40	50	30

^a The water depth refers to mean water level

The thickness of marine growth may be assumed to increase linearly to the given values over a period of 2 years after the structure has been placed in the sea.

Unless more accurate data are available, the roughness height may be taken as 20 mm below + 2 m. The roughness should be taken into consideration when determining the coefficients in Morison's equation.

The weight of marine growth is classified as a variable functional action. Unless more accurate data are available, the specific weight of the marine growth in air may be set equal to 13 kN/m^3 .

If marine growth exceeds the values for which the installation is documented, cleaning may be omitted if a new analysis shows that the structure has sufficient strength.

6.6.2 Water level, settlements, subsidence and erosion

When determining water level in the calculation of actions, the tidal water and storm surge shall be taken into account according to Table 4. Uncertainty of measurements, subsidence in the reservoir or settlement of the structure and possible erosion shall be considered. Calculation methods that take into account the effects that the structure and adjacent structures have on the water level shall be used.

The possibility of, and the consequences of, subsidence of the seabed as a result of changes in the subsoil and in the production reservoir during the service life of the installation, shall be considered. Reservoir settlements and subsequent subsidence of the seabed should be calculated as a conservatively estimated mean value.

The tidal data may be taken from Gjevik, Nøst and Straume (1990).

6.6.3 Appurtenances and equipment

Hydrodynamic actions on appurtenances (e.g. anodes, fenders) shall be taken into account, when relevant.

6.7 Combinations of environmental actions

Characteristic values of individual environmental actions are defined by annual exceedance probabilities of 10^{-2} (for ULS) and 10^{-4} (for ALS). The long-term variability of multiple actions is described by a scatter diagram or joint density function including information about direction. Contour curves or surfaces for more than two environmental parameters, can then be derived which give combination of environmental parameters which approximately describe the various actions corresponding to the given exceedance probability.

Alternatively, the exceedance probabilities can be referred to the action effects. This is particularly relevant when the direction of the action is an important parameter.

For fixed installations colinear environmental actions are normally most critical, and the action intensities for various types of actions can be selected to correspond to the exceedance probabilities given in Table 4.

For other installations action combinations which involve a large difference in action direction needs to be addressed.

In a short-term period with a combination of waves and fluctuating wind, the individual variations of the two action processes can be taken to be uncorrelated.

Table 4 – Combination of environmental actions with expected mean values and annual probability of exceedance 10^{-2} and 10^{-4}

Limit state	Wind	Waves	Current	Ice	Snow	Earthquake	Sea level ^a
Ultimate Limit State	10^{-2}	10^{-2}	10^{-1}	-	-	-	10^{-2}
	10^{-1}	10^{-1}	10^{-2}	-	-	-	10^{-2}
	10^{-1}	10^{-1}	10^{-1}	10^{-2}	-	-	m
	-	-	-	-	10^{-2}	-	m
Accidental Limit State	-	-	-	-	-	10^{-2}	m
	10^{-4}	10^{-2}	10^{-1}	-	-	-	m*
	10^{-2}	10^{-4}	10^{-1}	-	-	-	m*
	10^{-1}	10^{-1}	10^{-4}	-	-	-	m*
	-	-	-	10^{-4}	-	-	m
	-	-	-	-	-	10^{-4}	m
^a m - mean water level m* - mean water level, including the effect of possible storm surge Seismic response analysis should be carried out for the most critical water level.							

7 Deformation actions

7.1 General

Deformation actions are actions caused by deformations, imposed on the structure. They may be caused by the structure's function or the surrounding environmental conditions, or by construction processes.

7.2 Temperature actions

Structures shall be designed for the most extreme temperature differences they may be exposed to. This for instance applies to

- a) storage tanks,
- b) structural parts that are exposed to radiation from the top of a flare boom. One hour mean wind with a return period of 1 year may be used to calculate the spatial flame extent and the air cooling in the assessment of heat radiation from the flare boom,
- c) structural parts that are in contact with pipelines, risers or process equipment.

The ambient sea or air temperature is calculated as an extreme value with an annual probability of 10^{-2} .

Unless more accurate measurements or calculations are carried out, air and sea temperatures may be taken from Figure 8, Figure 9 and Figure 10. Sea temperature also varies with depth. The local air temperature may be higher as a result of sun radiation. During fabrication of the structure, all dimensions should be related to a reference temperature.

Eriksrød and Ådlandsvik (1997) give data for sea temperatures at the sea floor that can be used in an early design phase.

7.3 Actions due to fabrication

During design, reasonable tolerances shall be assumed, and account shall be taken for possible actions that may arise from forces introduced to compensate geometrical mismatch, shrinkage forces of concrete or welding. The effects of errors (e.g. geometric deviations or defects exceeding the tolerance limits) should be considered by the person responsible for the design.

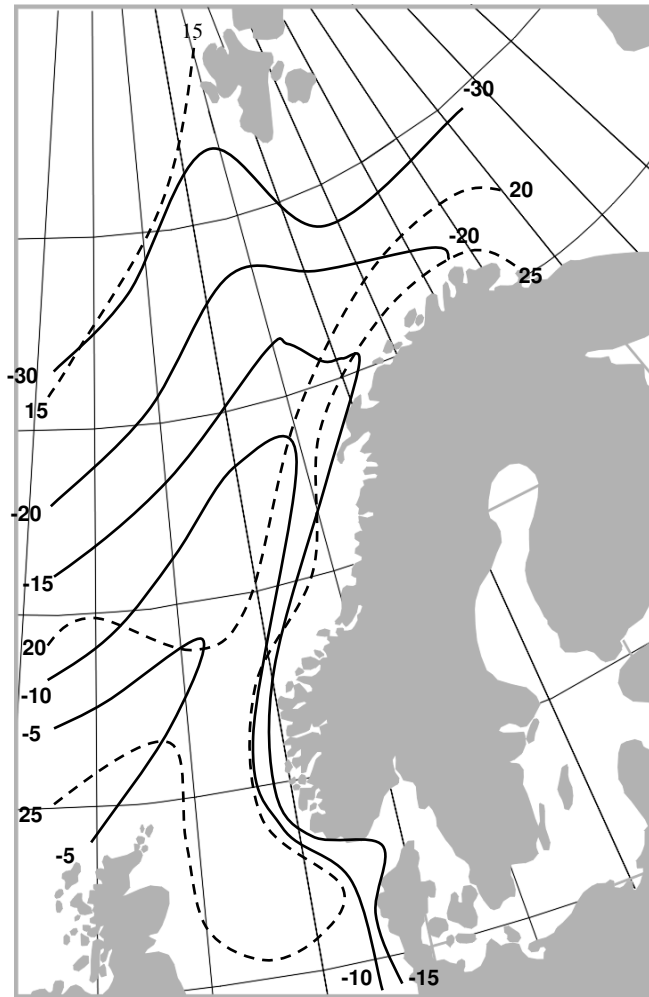


Figure 8 - Highest and lowest air temperature with an annual probability of exceedance of 10^{-2} (the temperatures are given in °C)

7.4 Actions due to settlement of foundations

Effects of uneven settlements of the foundation shall be considered. Actions on the structure from risers and drill-string as a result of foundation settlements should be considered. Local reaction actions on the structure during installation due to uneven seabed or boulders shall be considered.

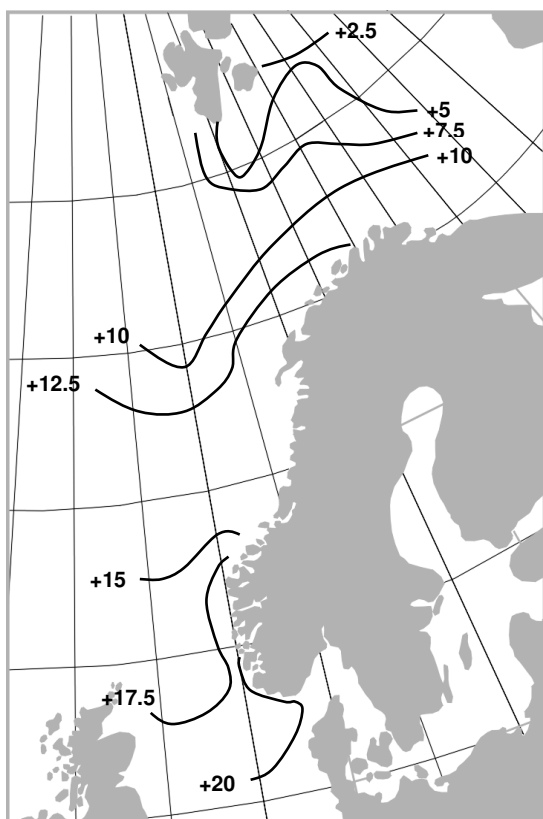


Figure 10 - Lowest surface temperature in the sea with an annual probability of exceedance of 10^{-2} (the temperatures are given in °C)

8 Accidental actions

8.1 General

- establish exceedance diagram for the action on each component;
- allocate a certain portion of the reference exceedance probability (10^{-4}) to each component;
- determine the characteristic action for each component from the relevant action exceedance diagram and reference probability.

Alternatively, the following, more refined consideration of risk may be used to determine the accidental action:

- component (i) is assumed to be designed for an accidental action with an exceedance probability of p_i for that component;
- estimate the probability of total loss due to failure of component (i), implied by the residual risk associated with the accidental action;
- estimate the total probability of failure (p_f) associated with the given accidental action on all components
- compare p_f with the target level;
- re-allocate p_i 's in order to get a more optimal design, while complying with the target level.

If the accidental action is described by several parameters (e.g. heat flux and duration for a fire, pressure peak and duration for an explosion) design values may be obtained from the joint probability distribution by contour curves, see 6.2.2.3. However, in view of the uncertainties associated with the probabilistic analysis, more pragmatic approach would normally suffice.

Relevant accidental actions and their magnitude should be determined on the basis of risk analyses and relevant accumulated experiences. With regard to the planning, implementation, use and updating of such analyses, reference is made to the PSA's Management Regulations.

Since a large amount of scenarios associated with each type of accidental action can be envisaged, a focus on those that may have impact on design is necessary.

Model tests or in-service observations may be necessary to study the accidental actions.

8.2 Fires and explosions

8.2.1 General

The principle fire and explosion events are associated with hydrocarbon leakage from flanges, valves, equipment seals, nozzles, etc.

The following types of fire scenarios should, among others be considered:

- a) burning blowouts in wellhead area;
- b) fire related to releases from leaks in risers; manifolds, loading/unloading or process equipment, or storage tanks (including jet fire and fire ball scenarios);
- c) burning oil on the sea;
- d) fire in equipment or electrical installations;
- e) fire on helicopter deck;
- f) fire in living quarters;
- g) pool fires on deck or sea.

The following types of explosions should be considered:

- a) ignited gas clouds;
- b) explosions in enclosed spaces, including machinery spaces and other equipment rooms as well as oil/gas storage tanks.

Structural layout should be selected so as to limit the effect of fire and explosion, e.g. by using appropriately located and sized fire and blast walls.

8.2.2 Fires

The fire action intensity may be described in terms of thermal flux as a function of time and space or, simply, a standardised temperature-time curve for different locations.

The fire thermal flux may be calculated on the basis of the type of hydrocarbons, release rate, combustion, time and location of ignition, ventilation and structural geometry, using simplified conservative semi-empirical formulae or analytical/numerical models of the combustion process.

The first step in determining heat flux actions is to establish the geometry of the flame to check whether the object is engulfed by the flame or not. Prediction of heat flux for engulfed structures is difficult.

The heat flux may be determined by empirical, phenomenological or numerical method, see SCI, 1992 - 1999.

Empirical models with reasonable accuracy exist for pool fires, cloud fires and fireballs. For some types of fires in topside areas and on sea, direct data for heat fluxes on relevant target surfaces are available, e.g. SCI, 1992-1999. Their shortcomings outside the validation range should be observed.

In principle the numerical methods provide a rigorous description of the relevant turbulence and combustion processes as well as flame radiation. Limitations in current models, however, make it necessary carefully validate them against experimental data.

The effect of each fire scenario should be determined on the basis of

- heat balance (incident radiant and convective heat flux is balanced by heat radiated, convected and conducted away from the surface),
- heat flow characteristics of the structure,
- mechanical and thermal properties at elevated temperatures,
- properties of active and passive fire protection systems, where applicable.

For objects engulfed by the flame the heat convected away can be neglected. For non-engulfed objects the incident convection is usually negligible.

The temperature in the structure may be calculated by finite difference method or analytical methods. The latter is limited to one-dimensional passage of the heat into the structure.

More detailed information about emissivity constants and thermal parameters may be found in ENV 1991-2-2 and associated specific standards for different materials.

Fire mitigation should be considered by detection, warning and shut-down system as well as fire protection systems. Fire protection may be obtained by passive fire protection materials and a fire protection such as water deluge, foam or, in some instances, fire suppressing gas.

Extinguishing fires may lead to use of large quantities of water. The consequences of the corresponding increase of weight should be taken into account.

Further details about determination of action effect (damage) may be found in NORSOK design standards for the relevant type of material.

8.2.3 Explosions

Explosions can cause two types of action, namely overpressure and drag.

The overpressure action due to expanding combustion products may be described by the pressure variation in time and space. It is important to ensure that the rate of rise, peak overpressure and area under the curve are adequately represented. The spatial correlation over the relevant area that affects the action effect should also be accounted for. Equivalent constant pressure distributions over panels could be established based on more accurate methods.

Drag action is caused by blast generated wind, and is a function of gas velocity squared, gas density, coefficient of drag, and the exposed area of the object. Drag action may affect piping, equipment and other items.

Main fire walls in enclosed spaces should at least resist an explosion pressure of 70 kN/m^2 with a duration of 0,2 s and an overpressure rise time of 0,1s.

The explosions pressure and wind drag depend upon the type of gas, ratio of gas in mixture, degree of mixing, location and time of ignition, the size and geometry of the room or confinement as well as size and location of vent and relief area. The latter two features can be influenced by design and provide means of mitigation.

Overpressure may be determined by semi-empirical methods established by experiments or theoretical models of the combustion and venting process (see SCI, 1992-1999) with supplementary technical notes.

Caution should be exercised in applying empirical methods outside the parameter range of underlying experimental data.

Numerical models solve the underlying equations describing gas flow, turbulence and combustion processes. The complexity of the explosion phenomenon requires the numerical methods to be validated by experimental results and used with caution.

The damage due to explosion should be determined with due account of the dynamic character of the action effects. Simple, conservative single degree of freedom models may be applied. When necessary, nonlinear time domain analyses based on numerical methods like the FE method should be applied.

8.2.4 Combined fire and explosion effects

Accidental actions, which are combinations of fire and explosion events, should be considered with an annual probability of 10^{-4} for the joint event.

Fire and explosion events that result from the same scenario of released combustibles and ignition should be assumed to occur at the same time, i.e. to be fully dependent. The fire and blast analyses should be performed by taking into account the effects of one on the other.

The damage done to the fire protection by an explosion preceding the fire, should be considered.

8.3 Impact actions

8.3.1 General

Impact actions are characterised by kinetic energy, impact geometry and the relationship between action and indentation. Impact actions may for instance be caused by

- a) vessels in service to and from the installation (including supply vessels),
- b) tankers loading at the field,
- c) ships and fishing vessels passing the installation,
- d) floating installations, e.g. flotels,
- e) aircraft on service to and from the field,
- f) falling or sliding objects,
- g) fishing gear,
- h) ice-bergs or ice.

Structural layout should be selected so as to limit the effect of impacts. Particular attention should be paid to protecting critical component such as risers.

ALS design checks should be made with impact events corresponding to exceedance probabilities of 10^{-4} .

8.3.2 Vessel collisions

The collision energy can be determined on the basis of relevant masses, velocities and directions of ships or aircraft that may collide with the installation. When considering the installation, all traffic in the relevant area should be mapped and possible future changes in vessel operational pattern should be accounted for. Design values for collisions are determined based on an overall evaluation of possible events.

Experience has shown that the design should take into account collisions by vessels intended for regular service inside the safety zone. The velocity can be determined based on the assumption of a drifting ship, or on the assumption of erroneous operation of the ship.

In the early phases of platform design, the mass of supply ships should normally not be selected less than 5 000 tons and the speed not less than 0,5 m/s and 2 m/s for ULS and ALS design checks, respectively. A hydrodynamic (added) mass of 40 % for sideways and 10 % for bow and stern impact can be assumed. Further information about collision analysis may be found in NORSOK N-004 and Veritec (1988).

The most probable impact locations and impact geometry should be established based on the dimensions and geometry of the structure and vessel and should account for tidal changes, operational sea-state and motions of the vessel and structure which has free modes of behaviour. Unless more detailed investigations

are done for the relevant vessel and platform, the impact zone for supply vessels should be considered to be between 10 m below low astronomical tide and 13 m above high astronomical tide.

Impact scenarios should be established representing bow, stern and side impacts on the structure as appropriate. If a central impact (impact action through the vessel's centre of gravity) is physically possible, this impact situation should be analysed.

The possibility of a second, less intensive impact on the damaged structure should be assessed. In this connection the possible change of floating position caused by the initial damage, should be assessed.

If a flotel or other installation is to be positioned close to the structure, special evaluations shall be carried out as to whether these installations may inflict collision actions on the other installation.

When the duration of the collision is short compared with the periods governing the motion and the rate of loading is relatively small, the damage caused in the collision in structures with free modes (see 10.1) may be determined in two steps, as described below.

First the distribution of impact energy between kinetic rotation and translation energy and deformation energy, can be determined by momentum and energy considerations. Then local damage to vessel and installation can be determined so that the energy absorbed by the two structures corresponds to the energy that is to be absorbed as deformation energy.

If the impact duration is long compared with the relevant local or global periods of structural vibration, structural analysis to determine the energy absorption and damage can be done by a quasi-static method of analysis. Otherwise, a dynamic structural analysis should be carried out. This analysis can be based on action - indentation curves obtained by laboratory tests and analysis, as outlined in NORSOK N-004.

8.3.3 Dropped objects

Actions due to dropped objects should for instance include following types of incidents:

- a) dropped cargo from lifting gear;
- b) falling lifting gear;
- c) unintentionally swinging objects;
- d) loss of valves designed to prevent blow-out or loss of other drilling equipment.

The impact energy from the lifting gear can be determined based on lifting capacity and lifting height, and on the expected weight distribution in the objects being lifted.

Unless more accurate calculations are carried out, the action from falling objects on to the deck may be based on the safe working action for the lifting equipment. This action should be assumed to be falling from the lifting gear from the highest specified lifting height and at the most unfavourable place. Sideways movements of the dropped object due to possible motion of the structure and the crane hook should be considered.

The trajectories and velocity of objects dropped in water should be determined on the basis of the initial velocity, impact angle with the water, effect of water impact, possible current velocity and the hydrodynamic resistance. It is non-conservative for impacts in shallow water depths to neglect the effect of water impact. A typical trajectory for slender objects (pipes) is straight motion until it has reached the maximum horizontal excursion. Then it starts to fall, exhibiting horizontal oscillations. The possibility of a straight trajectory for pipes with a rotation about the longitudinal axis should be considered.

The impact effect of long objects such as pipes should be subject to special consideration.

Relevant impact situations can be determined by the operating area of the lifting equipment and the relevant lifting arrangement.

Whether the equipment on deck may fall down should be considered. Damage caused by subsequent actions should also be considered.

Similarly it should be considered what damage possible fender systems would cause if they should fall down.

NORSOK U-001 provides values for accidental actions on subsea installations. These values may be used in evaluations during an early design phase.

8.3.4 Helicopter impacts

If specific information about the weight distribution of the undercarriage of a helicopter is not available, the helicopter deck should be designed on the presumption that any point on the deck may be subjected to a single action equal to 75 % of the total weight of the heaviest helicopter used. The single action can be assumed evenly distributed over an area of 0,09 m². Reference is made to NCAA Regulations. The main structure below the helicopter deck should be designed for an action equal to 3 times the total weight of the heaviest helicopter used. The normal weight distribution of the helicopter on the undercarriage may be used. It should be assumed that the helicopter is placed in the most unfavourable position.

8.4 Buoyancy loss due to subsea gas blow-out

For floating structures it should be considered whether a subsea gas blow-out can cause loss of buoyancy or a displacement of significance to the stability, or motions of significance to the anchoring system or to possible contact with other installations.

8.5 Loss of heading control

For units normally operated with heading control, either by weather vaning or thruster assistance, the effect of loss of the heading control shall be evaluated.

8.6 Abnormal variable actions

Changes in intended pressure differences or buoyancy caused for instance by defects in or wrong use of separation walls, valves, pumps or pipes connecting separate departments as well as safety equipment to control or monitor tank pressure, shall be considered.

Unintended distribution of ballast due to operational or technical faults should also be considered.

8.7 Actions on a floating structure in damaged condition

Floating structures which experience buoyancy loss will have an abnormal floating position. The corresponding abnormal variable and environmental actions should be considered.

Adequate global structural strength (according to the second step of the ALS check) should be documented for abnormal floating conditions considered in the damage stability check.

8.8 Combination of accidental actions

When accidental actions occur simultaneously, the probability level (10^{-4}) applies to the combination of these actions. Unless the accidental actions are caused by the same phenomenon (like hydrocarbon gas fires and explosions), the occurrence of different accidental actions can be assumed to be statistical independent.

While in principle, the combination of two different accidental actions with exceedance probability of 10^{-2} or one at 10^{-3} and the other at a 10^{-1} level, correspond to a 10^{-4} event, individual accidental actions at a probability level of 10^{-4} , commonly will be most critical.

9 Action combinations

9.1 Normal operation

9.1.1 General

Table 5 shows which actions that should be combined in different limit states.

Combinations of environmental actions are given in 6.7, and combination of accidental actions is given in 8.8.

Table 5 – Characteristic actions and action combinations

	TEMPORARY CONDITIONS					NORMAL OPERATIONS				
	Serviceability limit state	Fatigue limit state	Ultimate limit state	Accidental limit state		Serviceability limit state	Fatigue limit state	Ultimate limit state	Accidental limit state	
				Abnormal effect	Damaged conditions				Abnormal effect	Damaged condition
Permanent actions	EXPECTED VALUE									
Variable functional actions	SPECIFIED VALUE									
Environmental actions	Dependant on operational requirements	Expected action history	Value dependent on measures taken			Dependent on operational requirements	Expected action history	Annual probability of exceedance = 10 ⁻²	Annual probability of exceedance = 10 ⁻⁴	Annual probability of exceedance = 10 ⁻²
Deformation actions	EXPECTED EXTREME VALUE									
Accidental actions	Not applicable			Dependent on measures taken	Not applicable				Annual probability of exceedance = 10 ⁻⁴	Not applicable

9.1.2 Static and dynamic pressures in tanks

When combining static and hydrodynamic tank pressures, appropriate account of relevant tank filling and acceleration levels for different tanks should be accounted for.

The pressure height, h_D , and pressure, p_0 , associated with pumping operations (see 5.4.1) could be neglected if such operations are documented not to occur at the same time as extreme sea actions.

9.1.3 Variable and environmental actions in combination with accidental actions

Account shall be taken of the permanent and variable actions that might reasonably be present at the time of the accidental event.

When environmental and accidental actions occur simultaneously, the given probability level applies to the combination of these actions. Unless environmental actions contribute to the occurrence of accidental actions these two actions can be assumed to be statistically independent. The expected environmental action occurring together with the 10^{-4} accidental can be neglected unless the accidental action is initiated by the environmental action.

The damaged structure, resulting from an accidental action event, shall be able to resist relevant permanent and variable actions in an environmental condition corresponding to annual exceedance probability of 10^{-2} .

9.2 Temporary conditions

Temporary conditions may refer to conditions during fabrication, installation or use.

Environmental and accidental actions in temporary phases depend upon the measures taken.

Precautions may be taken to ensure that maximum tank pressures, associated with pressure testing, do not occur during maximum environmental conditions.

Precautions may be taken in connection with environmental actions, by carrying out the operation during a period when the environmental conditions is ensured to be acceptable.

If no precautions can be taken with respect to environmental conditions the characteristic action should be defined by the "Normal operation" in Table 5. For operations performed in a particular season, the exceedance probability could be taken to refer to the particular season. However, the period should not be considered to be less than 3 months.

For operations lasting more than the time (3 days) for reliable weather forecast, and there is no danger of injury or damage to people or to the environment or of major economic consequences, an environment condition corresponding to a one year return period may be used.

For operations lasting more than 3 days, but where it is possible, within 24 h to bring the structure into a condition which will resist actions specified according to the procedure given above, the structure may be

designed for a smaller environmental action. The operation shall be discontinued if the weather forecasts for the next 3 days indicate values in excess of the planned environmental condition.

For operations with duration up to 3 days can be planned with a smaller environmental action than given above. The operation may then be commenced when forecasts provide adequate certainty that the planned condition is not exceeded.

Accidental actions and possible abnormal operational conditions in temporary phases are to be determined by risk analysis.

In temporary phases of short duration, controls may be normally implemented to ensure that accidental actions are of negligible magnitude.

10 Action effect analyses

10.1 General

Action effects, in terms of motions, displacements, or internal forces and stresses of the structure, should be determined with due regard for

- the spatial and temporal nature including,
- possible non-linearities of the action,
- dynamic character of the response,
- the relevant limit states for design check,
- the desired accuracy in the relevant design phase.

While permanent - , functional - , deformation - and fire actions generally can be treated by static methods of analysis, environmental (wave and earthquake) actions and certain accidental actions (e.g. impacts, explosions) may require dynamic analysis. Inertia and damping forces are important when the frequency of steady-state actions are close to natural frequencies or when transient actions occur.

The global behaviour of offshore structures may be characterised by the motion characteristics for each rigid body mode under wave frequency actions. Restrained modes of behaviour typically have a natural frequency above the wave frequency, and free modes have a natural frequency in or below the wave frequency range.

Table 6 characterises some concepts.

Table 6 – Characterisation of global behaviour

Type of structure	Translation			Rotation		
	Vertical (Heave)	Horizontal (Surge)	Horizontal (Sway)	Vertical plane (Pitch)	Vertical plane (Roll)	Horizontal plane (Yaw)
Fixed steel (jacket, jack-up, subsea, template)	R	R	R	R	R	R
Fixed concrete (framed)	R	R	R	R	R	R
Tension-leg, with three legs or more	R	F	F	R	R	F
Semi-submersible ^a	F	F	F	F	F	F
Ship ^a	F	F	F	F	F	F
Spar buoy ^a	F	F	F	F	F	F
Articulated tower	R	(F)	(F)	F	F	(R)
^a With catenary mooring or dynamic positioning system. F: free R: restrained mode of motion						

A global wave motion analysis is necessary for structures with at least one free mode. For fully restrained structures a static or dynamic wave-structure-foundation analysis is required.

Uncertainties in the analysis model are expected to be taken care of by the partial safety factors. If uncertainties are particularly high, conservative models shall be selected.

Simplified methods may be applied if they are properly documented and they provide conservative results.

If analytical models are particularly uncertain, the sensitivity of the models and the parameters utilised in the models shall be examined. If geometric deviations/imperfections have a significant effect on safety, conservative geometric parameters shall be used in the calculation.

In the final design stage theoretical methods for prediction of important responses of any novel system should normally be verified by appropriate model tests, see 10.2.6.

Earthquake actions need only be considered for restrained modes of behaviour.

10.2 Global motion analysis

10.2.1 Purpose

The purpose of a motion analysis of structures with at least one free mode according to Table 6, is to determine displacements, accelerations, velocities and hydrodynamic pressure relevant for the action on the hull, superstructure, riser and mooring system, as well as relative motions (in free modes) needed to assess airgap and green water requirements. Excitation by waves, current and wind should be considered.

10.2.2 Environmental and operational conditions

The response analysis shall be carried out with due regard for combined environmental conditions, e.g. wave, current and wind intensity and direction. Attention should be paid to the fact that current actions may dominate the hydrodynamic actions for some concepts. If joint statistical data for the site or area is not available, conservative assumptions shall be made, including consideration of unidirectional versus multidirectional environment.

Design actions and action effects may depend on operational practice. The assumptions made in the analysis regarding operations shall be consistent with relevant instructions and limitations for safe operation. This involves the magnitude and distribution of deck action. For buoyant structures the ballast condition, the floating position (e.g. draught, tilt) and mooring tension, are of concern. For ships, heading direction is also an important issue. The margin to cover the variability of ship heading with respect to the direction of wind, current, wind-waves and swell, should depend on the directional control (active and passive) used.

Possible lack of directional stability of ships or barges that may cause excessive yaw motions ("fish-tail" behaviour) and, hence, roll and heave motions should be noted in connection operational control.

When calculating the action effects due to waves, floating platforms may be assumed to be ballasted to an even keel in a 100 years mean wind condition.

This is conditional upon availability of necessary equipment, adequate procedures and sufficient time for correcting the floating condition during change of the environmental condition, i.e. direction and speed of wind.

Design analyses shall not be based on operational precautions of the mooring lines during operation.

10.2.3 Static and mean response analysis

Static analyses are carried out to determine

- the static equilibrium position of the platform under gravity and buoyancy actions in still water. A static analysis should be performed for each action condition to be analysed, considering the total platform weight, ballast, buoyancy (displacement), riser and mooring tensions, and hook actions,
- the position of the platform under mean wind force, current force and steady wave drift force, based on the static equilibrium condition,
- overturning moment acting on (floating) platforms with six free modes subject to sustained wind actions in connection with stability check.

The determination of a mean or equilibrium position due to steady environmental actions is the basis for proceeding with a dynamic analysis. The mean position should be determined with appropriate values of the sustained actions, in view of their correlation with the fluctuating actions in the dynamic analysis.

10.2.4 Dynamic analysis

10.2.4.1 Dynamic model

The dynamic equation of equilibrium is formulated in terms of

- excitation actions,
- inertia actions,
- damping actions,
- restoring actions.

relating to the hull, positioning system and risers.

Possible excitation, inertia, damping and restoring actions by the riser and mooring system could be introduced in a simplified fashion for 6 degrees of freedom system models (uncoupled analysis) or more exactly in a coupled platform/mooring/riser analyses with more degrees of freedom, see 10.2.5.

In connection with dynamic response analyses excitation actions may conveniently be categorised as

- WF actions (affecting all 6 degrees of freedom),
- HF wave actions (affecting restrained modes of motion),
- LF (or slow drift) actions (affecting primarily horizontal motions but sometimes also vertical modes),
- wind gust actions (affecting roll, pitch as well as horizontal motions),
- transient wave slamming, run-up and ringing actions.

HF or LF wave actions are normally an order of magnitude smaller than wave frequency actions. Their effect may be significant if wave frequency actions are minimised by design, or when they are close to fundamental frequencies of vibrations of the system.

Example of resonance of (lightly damped) modes of motion may for instance include

- roll of a barge/ship or a spar with a low metacenter height, (free mode),
- heave of a Spar buoy (free mode),
- vortex induced motions for deep draught platforms,
- surge, sway and yaw motion of a catenary moored floating production system (free mode),
- internal U-tube resonances (local dynamic mode),
- ballast or cargo tank sloshing modes (local dynamic mode),
- heave/pitch/roll resonance of TLPs (restrained mode),
- moon pool motion.

The transient response caused by slamming actions would generally be associated with structural dynamic effects.

Inertia actions are related to the hull, mooring and riser mass, variable mass (e.g. ballast), and added mass due to of the surrounding water.

Damping has linear contributions from wave radiation and wave drift damping. Viscous damping actions are associated with the hull, risers, and mooring systems.

Assessment of damping is uncertain and shall be conservatively estimated.

The effect of possible thrusters in terms of restoring actions and possible damping should be included. Also, the damping effect of free surface in tanks shall be included in this connection.

Viscous drag actions on slender bodies can be estimated by Morison's equation, and should be estimated separately or combined (by using relative velocity) for wave - and low frequency motions by using a proper drag coefficient.

The current gives rise to damping as well as excitation actions. For resonant motion behaviour it is recommended that analyses are carried out with different values of current velocity, spanning from zero up to design values and that the most conservative results are designed for.

Restoring forces are contributed by the buoyancy effect on the vessel, and by the catenary or taut mooring system as well as by the riser systems. Long-term change of restoring properties of synthetic ropes should be accounted for. The effect of catenary mooring on heave restoring actions is normally negligible. Tether elastic deformations can also normally be neglected in determining the behaviour of free modes of TLPs.

10.2.4.2 Frequency domain analysis for a short term period

A frequency domain analysis for a short term period may be carried out to determine wave frequency motions and mooring actions as a function of wave frequency at a suitable mean condition. The response spectrum obtained together with the assumption of Gaussian process may be used to estimate the response statistics valid for the actual condition.

The commonly used frequency domain solution techniques require linear equations of motion. This is inconvenient when modelling velocity squared drag actions, time varying geometry, horizontal restoring actions and variable water surface elevation, since these effects are nonlinear. In many cases, these nonlinearities can be satisfactorily linearized about some operating point or by using an equivalent linearization technique.

When determining extreme values for cases where nonlinear actions or action effects could be important, linearized solutions should be used with care.

10.2.4.3 Time domain analysis for a short-term period

Time domain analysis means numerical integration of the equations of motion allowing the inclusion of all system nonlinearities, using direct step-by-step integration techniques.

Time domain solution methods should generally be used in case of significant nonlinear effects. Time domain analysis is also normally required to determine the transient response after slamming, ringing events, as well as mooring component failure. Such analysis allows handling for instance of drag actions which are nonlinear functions of the fluid velocity, finite amplitude and finite wave amplitude effects, and nonlinear geometrical and material effects associated with the station keeping system.

Time domain methods are usually used for extreme condition analysis, but are normally not required for fatigue analysis or analysis of more moderate conditions where the more efficient linearized analysis provide sufficient accuracy. However, time domain analysis of fatigue action effects may be required in connection with local splash zone actions.

Time domain analysis may be carried out for some sea-states and generalised to other conditions, e.g. by using an equivalent sea state dependent transfer function.

Periodic analysis must be carried far enough to achieve steady state. Irregular analysis shall be carried far enough to achieve stationary statistics.

When the analysis is performed in a random sea consideration should be given to the frequency dependency of the added mass and damping coefficients.

A wave spectrum is used to generate random time series when simulating irregular wave kinematics. The linear and nonlinear wave exciting actions are both represented in the form of time histories derived from the wave time history. If time-domain simulations are carried out to determine air gap or green water or other action effects which are sensitive to the non-Gaussian character of real extreme seas, this feature should be accounted for.

The time series resulting from the time domain analysis of nonlinear systems will generally be non-Gaussian. To limit the statistical uncertainty, sufficiently long time series should be generated, and statistical fitting techniques should be used to determine the expected maximum response.

A strategy to improve the computational efficiency in time domain simulation of a complex model exposed to irregular excitation is to simulate critical events (i.e. slam, ringing events) with the refined model for a time

duration identified by a simplified approach. A fundamental requirement is that the relevant events shall be captured by the identification procedure. This technique should therefore be applied with care.

Guidelines for implementation of time series analyses can be found in SNAME 2002.

The short term time domain analysis may be used in conjunction with a smooth joint probability distribution for H_s and T_p as well as other possible parameters.

10.2.4.4 Low-frequency analysis

A low-frequency dynamic analysis is to be carried out to determine the actions and motions due to combined actions such as

- second-order wave actions,
- current viscous actions,
- vortex induced motions of deep draught floating platforms,
- fluctuating and sustained wind actions,
- non-linear mooring restoring actions.

Appropriate methods of analysis are to be used.

All relevant sustained horizontal actions including current, as well as the effect of wave frequency motions are to be included in the analysis.

Frequency range is to include all low-frequency cyclic actions that may excite platform resonant motions.

Appropriate methods for estimation of damping of the low frequency motion analysis are to be used. If important, coupled wave-and low frequency motions must be considered.

The extreme LF wave- and wind-response should therefore be determined with due consideration of WF response as well as the mean environmental actions. This may be conveniently achieved by a time-domain approach with a sufficiently long time period. Alternatively, especially in an early design stage, the WF- and LF-responses may be determined separately in the frequency domain, and combined, as for instance described in 10.2.4.5.

The low-frequency wind-induced forces can be modelled as a Gaussian process when the sustained wind dominates over gust wind. If the wind-induced motion response is approximated by a Gaussian process, the characteristic largest motion amplitude for this process may be calculated as $X_{LF1c} = \sigma_{X-LF1} \sqrt{2 \ln N_{LF1}}$ where σ_{X-LF1} is the standard deviation and N_{LF1} is the mean value of zero up-crossings in the period considered (N_{LF1} number of motion cycles in the period).

Low-frequency wave-induced motions are generally non-Gaussian due to the inherent non-linearities, and dependent upon the wave-frequency motions. This analysis should be carried out using restoring characteristics of the mooring lines that correspond to the mean environmental actions. Low-frequency damping is a critical parameter and should be estimated with caution. The largest LF-motion amplitude should be estimated based on a suitable method. The motions follow a Rayleigh or an exponential distribution for the case of low and high system damping, respectively. Normally the actual distribution is between the two special cases. The characteristic maximum may be conservatively estimated by assuming an exponential distribution $X_{LF2c} = \sigma_{X-LF2} \cdot \ln N_{LF2}$. A more realistic estimate can be obtained by Stansberg's method, see Stansberg (1992).

Motion-induced tension in mooring lines will in general be non-Gaussian due to nonlinear load-displacement relationship of mooring lines.

The ratio between the maximum value and standard deviation of motions or mooring line tension used in final design should be documented by time series analysis, model tests or field measurements, or by calibrated, simplified methods.

Low-frequency wind- and wave-induced motions in the same direction can be assumed to be independent and combined by a square root of sum of square approach.

$$X_{LF} = \sqrt{X_{LF1}^2 + X_{LF2}^2} \quad (14)$$

10.2.4.5 Combined extreme wave- and lowfrequency action effects

The characteristic value X_c , of the combined wave frequency X_{WFC} , and total low frequency motions in the same direction depends upon the relative magnitude of low frequency wind- and wave-induced actions, and system characteristics. The wave frequency response X_{WF} , is determined by using restoring characteristics at a platform location determined by the mean and maximum LF environmental actions. While the WF motions can be well described by a linear model, the dynamic line tension is nonlinear. However, it can be conservatively linearized. For a linear response with Rayleigh distribution: $X_{WFC} = \sigma_{X-WF} \sqrt{\ln N_{WF}}$. If the mean action effect X_{mean} is included, the total action effect may be expressed as:

$$X_c = \sqrt{X_{WFC}^2 + X_{LFC}^2 + 2\rho X_{WFC} X_{LFC}} + X_{mean} \quad (15a)$$

where ρ is a correlation coefficient. For horizontal motions of semi-submersibles, ρ will typically be between 0,2 and 0,4.

An alternative way of combining WF and LF action effects to obtain the total effect is:

$$X_c = \max[(X_{WF(max)} + X_{LF(sig)}); (X_{LF(max)} + X_{WF(sig)})] + X_{mean} \quad (15b)$$

where the indices "max" and "sig" refer to expected maximum and significant value, respectively. Eq. (15b) may be used for motion and mooring tension effects in catenary moored floating platforms. Significant values are then taken as $X_{LF(sig)} = 2\sigma_{X-LF}$

The mooring tension is calculated by account of dynamic mooring line effects by considering two equivalent conditions of excursions, namely $X_{mean} + X_{LF(sig)}$ and $X_{mean} + X_{LF(max)}$ and calculating the resulting maximum and significant mooring forces due to wave frequency excitation. The characteristic value is the maximum of the two.

If the mooring line has a quasi-static behaviour, the tension is directly calculated based on the maximum total excursion X given by Eq. (15a) or Eq. (15b).

10.2.4.6 Slamming, wave breaking and green water

If slamming or green water is likely to occur, calculation or model tests should determine the corresponding pressure actions. Slamming effects should be calculated by applying an appropriate dynamic structural model.

Slamming effects are of significance in combination with extreme first order wave frequency effects and should be evaluated in the time domain with due consideration of first order wave action effects.

Slamming actions may especially be of significance for ultimate and accidental limit states.

10.2.4.7 Fluid sloshing in tanks

Translational and angular motions, especially of ships, and floating platforms will generate motions within tanks. Depending upon the size of the tanks, the amount of water or oil and the motions of the structure at the resonant oscillation period of the fluid in the tank, a dynamic amplification of static pressure occurs, possibly combined with local impact actions. These sloshing actions may be significant from an ultimate strength and a fatigue standpoint.

A sloshing analysis shall be carried out in accordance with recognised calculation procedures, possibly combined with model tests. For ships the simplified methods given in the rules of recognised classification societies may be applied.

10.2.5 Dynamic system analysis for a short term period

10.2.5.1 General

Hull, risers and station-keeping system is an integrated dynamic system responding to environmental actions from wind, waves and current. This system may be analysed as a decoupled or coupled system.

10.2.5.2 Decoupled analysis

A decoupled analysis is carried out in two steps.

In the first step rigid body floater motions are computed considering static-, LF- and WF environmental actions. Some of the six degrees of freedom in this analysis may, however, be coupled between themselves. Risers and mooring system are represented by the static restoring force characteristics and a constant LF viscous damping. Assessment of the LF damping is crucial for the floater motion analysis. Contribution from current action on mooring lines and risers may be represented by a constant external action.

In the second step the floater motions and forces in case of TLP tension, computed in first step are applied as forced boundary displacements and forces, respectively on the slender structures. Wave and current actions on the slender structure can also be included. Forced WF floater motions are considered as dynamic excitation while LF and WF floater motions can alternatively be applied as forced dynamic excitation if considered to be of importance to the slender structure dynamics. A typical example is catenary mooring lines where WF line dynamics can be significantly influenced by the quasi-static LF tension variation. In addition, dynamic effects associated with LF as such should be considered for deepwater system.

The second step is the time consuming part of the decoupled analysis and is normally carried out for critical mooring lines and risers one by one. The computational flexibility contributes to efficient analyses and is the major advantage of the decoupled analysis.

10.2.5.3 Coupled analysis

Coupling effects between floater motions and slender structure response can be accounted for by including the floater force model in the slender structure model of the complete system including all mooring lines and risers.

The coupled approach requires significant computational efforts. As a compromise it can be proposed to apply a rather crude slender structure model in the coupled analysis still catching the main coupling effects, e.g. restoring, damping, mass. Detailed slender structure analysis can then be performed as in step two in the decoupled analysis based on the floater motions predicted in the coupled analysis. It can also be suggested to use a rather short coupled simulation for estimation of LF damping from mooring lines and risers. Damping estimate can then be used in a decoupled analysis.

Further guidance on coupled analysis may be found e.g. in DNV-OS-F201.

10.2.5.4 Mooring and riser analysis

Mooring lines and risers are commonly termed slender marine structures to emphasise similarities in system topology and global static- and dynamic behaviour. The main difference in mechanical properties affecting the global analysis is that risers are influenced by bending stiffness while mooring lines are not. It is therefore convenient to apply the same methodology for global analysis of risers and mooring lines. A FE approach is normally applied using beam and bar elements for efficient modelling and analysis of mooring lines and risers. For increased computational efficiency bar elements may also be used e.g. for flexible risers.

More detailed analyses of bending response close to the support is then calculated with the response obtained in the first analysis appropriately used as boundary condition.

An important feature of the analysis of slender structures is the treatment of nonlinearities due to hydrodynamic (drag) action and wave elevation variation effects, geometric stiffness, possible material nonlinearities and contact problems in terms of contact between slender structure and seafloor as well as hull.

The relative importance of these non-linearities are strongly system and excitation dependent, the first two non-linearities will, at least to some extent, always be present whilst the latter ones are more system specific non-linear effects. Material non-linearities will for example normally not be relevant for metallic tensioned risers while it is most important for non-bonded flexible pipes and synthetic mooring lines.

It should be noted that external and internal pressures in tubular members are not considered to be non-linear effect, as hydrostatic pressure will be handled by the effective tension concept in the analyses.

The response may be calculated by a (completely) linearized frequency or time domain approach.

An attractive approach when hydrodynamic action is the major nonlinear contributor, would be to linearize inertia, damping and stiffness at the static position while including the non-linear hydrodynamic action according to the Morison equation.

Analysis of catenary mooring systems can be carried out according to the technical requirements in the ISO 19901-7 or DNV-OS-E301. In any case the characteristic values of actions and the safety factors applied shall be in accordance with NORSOK N-001 and NMD regulations concerning anchoring and positioning on mobile offshore units (issued in 1987 with amendments issued in 1997).

10.2.5.5 Air gap analysis

Wave impact may, be permitted to occur on any part of a structure provided that it can be demonstrated that such actions are properly accounted for in the design and that the water does not threaten personnel's life, or damage pipes and other equipment which may lead to environmental damage. This means for instance that ULS and ALS strength requirements should be fulfilled for events with annual probabilities of 10^{-2} and 10^{-4} , respectively. The characteristic actions for such design checks then need to be determined. There is currently no air gap requirement as such.

However, due to the complexity and uncertainty associated with determining actions associated with waves hitting the platforms decks (e.g. in semi-submersibles, from below, and the non-linear relation between wave height and action effect) an air gap margin of 1,5 m on the 10^{-2} wave event, is recommended for fulfilling ULS criteria. The ALS criterion may be fulfilled by a positive air gap or by demonstrating survival of the platform subject to a 10^{-4} event. The deck structure adjacent to platform columns, however, needs to be designed to resist the possible pressure actions due to run-up along columns.

Air-gap considerations are also relevant for small water-plane area twin hull and catamaran ships as well as topside structures, located on platforms above the hull of a ship-shaped structure. For mono hull ships freeboard requirements relating to the hull, should be adhered to.

When assessing air gap the following effects shall, when relevant, be considered:

- water-level (including storm surges, astronomical tides, settlement, subsidence; and set-down for TLP);
- maximum/minimum operating draughts;
- static mean offset and heel angles;
- first order sea surface elevation including wave/structure interaction effects, i.e. wave enhancement;
- wave crest elevation including wave asymmetry (crest to trough ratio);
- wave frequency motions (in all 6 degrees of freedom);
- low frequency motion in heave, pitch and roll;
- effects of interacting systems (e.g. mooring and riser systems) (for buoyant structures).

Wave asymmetry may be accounted for by using a factor on first order wave response based on Gaussian sea states (or a higher order wave theory which is documented to yield reliable predictions). In absence of a detailed analysis, the crest height in deep water may be taken to be 0,6 times the wave height. For more detailed account reference is made to ISO 19901-1. The analysis of wave/structure interaction effects should be made according to 6.2.

Combination of different wave action effect can be carried out according to 10.2.4.4.

Analysis of interaction between hull and e.g. mooring and riser system is described in 10.2.5.2 to 10.2.5.4.

If the airgap analysis is subjected to significant uncertainties, it should be verified by model tests.

10.2.6 Long-term response analysis

The long-term distribution of action effect $x; f_x(x)$, can be combining the short-term action effects $f_{x|\underline{e}}(x|\underline{e})$, (in which $\underline{e}$ is the spectral parameters describing a sea-state) according to their probability of occurrence, as given by a scatter diagram $f_{\underline{e}}(\underline{e})$, as follows

$$f_X(x) = \int_E w(e) f_{X|E}(x|e) f_E(e) de \quad (16)$$

where $w(e)$ is a weight function defined as the average period in the long-term divided by average period of the given short-term sea-state.

10.2.7 Model testing

10.2.7.1 Purpose

In the conceptual design phase the primary objective of model tests in e.g. wave basin or wind tunnel, would be to confirm that no important feature (e.g. ringing, slamming or cork-screw motion of spar buoys with strakes) has been overlooked for temporary and in-place conditions.

In the final design phase the purpose would be to determine the responses (e.g. motions, mooring line tension, run-up, slamming) of a particular design. In this case it is important to predict the full-scale response of the actual system.

In general model tests may be used to verify methods for predicting systems response.

An important value of model tests is that the results are obtained without requiring many assumptions about the nature of the responses. This is generally not true of numerical models. However, model testing has its limitations too. There are numerous sources that can cause errors in model test results. Numerical predictions and model experiment results should be considered as being complimentary to one another.

Through careful interpretation, each of these results may be used to partially circumvent the limitations of the other.

10.2.7.2 Implementation of model test results

When implementing experimental test results into design, all relevant deviations between the model test and reality shall be considered. Such deviations may include

- scaling effects,
- model simplifications,
- limitations in testing facilities (e.g. finite dimensions; quality of waves, current and wind; wave absorption),
- simplifications and uncertainties regarding the data acquisition and processing,
- uncertainties with regard to long-term effects,
- the failure mode.

Statistical uncertainties with respect to limited sample maxima of test results are to be included in the determination of model responses.

Extreme values of action effects due to stochastic actions should be determined with due consideration of all information contained in the sample, with particular emphasis on the largest values of the sample. Extrapolation based on fitted distribution should be used.

The model test shall be planned, executed and documented in such a manner that they are repeatable.

NMD guidelines for wind tunnel tests shall be applied.

10.2.8 Full-scale measurements

Full-scale measurements may be used to update the response prediction of the relevant structure and to validate the methods for analysis of action effects for future design or redesign.

The updated analysis of the actual structure may have implications on operational requirements.

Such measurements tests should especially be devoted to actions and action effects which are difficult to simulate in model scale, i.e. associated with soil conditions and structures subjected to hydrodynamic actions involving both potential and viscous effects, as well as nonlinear effects such as ringing and air gap.

It is crucial to ensure adequate instrumentation to monitor environmental conditions during full-scale measurements.

10.3 Action effects in structures and soil/foundation

10.3.1 General

Displacements, forces or stresses in the structure and foundation, shall be determined for relevant combinations of actions by means of recognised methods, which take adequate account of the variation of actions in time and space, the motions of the structure and the limit state which is to be verified. Characteristic values of the action effects are to be determined.

Non-linear and dynamic effects associated with actions and structural response shall be accounted for, whenever relevant.

The stochastic nature of environmental actions should be adequately accounted for.

10.3.2 Action processes

10.3.2.1 Waves

Wave-frequency action effects may be determined by

- stochastic long-term set of sea-states,
- a set of stochastic short-term sea states,
- a design wave approach, specified by a regular long-crested wave with given height and length.

For structural design purpose, a combination of these approaches may be necessary to determine characteristic action effects for ULS design checks. In such a case, e.g. the stochastic approach may be used to determine the design wave parameters such that the design wave approach gives the same extreme response for representative response variables as the stochastic approach, with due consideration of wave steepness, as indicated in 6.2.

Maximum action effects for fully restrained structures are obtained by considering design waves with different headings and with extreme wave height and appropriate wave period which is varied within an appropriate interval. If structural dynamic effects are significant, this approach should be used with caution.

Actions on mono-hull ships and their effects can be appropriately calculated by a hierarchical set of models. The overall behaviour may be investigated by a relatively coarse FE mesh. This model may be used together with stochastic wave models to determine relevant action conditions for more detailed structural analyses. More detailed 3-D models may have to be used for particular hull sections like the turret area.

For floating space frame structures the analysis should include conditions with split forces (in transverse and longitudinal direction), torsion, horizontal shear and vertical shear due to acceleration of deck masses as well as vertical acceleration of deck masses. By using the design wave approach the simultaneity of global and local action effects may most easily be accounted for. The design wave approach for floating space frame structures may be calibrated by a stochastic approach by means of the sectional forces, e.g. heave, split, transverse shear.

However, for platforms with restrained modes that are affected by ringing, springing, whipping (slamming) a stochastic analysis of action effects would normally be necessary, see 10.2.4.2 and 10.2.4.3.

Wave actions due to low frequency excitation can normally be neglected for the hull, except for the fairlead, turret or other areas with mooring attachment.

Stress ranges for fatigue design should be determined for a representative set of sea-states using the long-term stochastic approach. A linear frequency domain approach is normally applicable to determine the response in each sea-state. Particular attention to nonlinear wave effects (e.g. a wave attenuation variation, and slamming) in the splash zone as well as possible sloshing in tanks, is, however, necessary.

Simplified approaches to determine fatigue action may be used if properly validated.

10.3.2.2 Wind

The fluctuating component of wind velocity is specified by a wind spectrum.

Stochastic wind analysis is described in 10.3.6.

10.3.2.3 Earthquake

Seismic actions may be described by ground response spectrum or time domain motion histories.

10.3.2.4 Accidental actions

Accidental actions are described in clause 8.

10.3.3 Modelling of structure or foundation

10.3.3.1 General

The model, or models, of the structure and soil/foundation shall be selected to adequately represent the simultaneous global and local action and provide the action effects needed for different limit states. For instance, the simultaneous effect of hydrostatic pressures or dynamic local pressures and the integrated (global) effect of stillwater and wave actions, need to be accommodated.

The effects of secondary and/or non-structural components are to be considered in an appropriate manner. As a minimum such components need to be modelled to account for the hydrodynamic forces and inertia forces due to motions.

Adequate structure/foundation interaction is to be included in global analyses. Catenary mooring may be represented by horizontal and vertical springs (or actions).

Tether forces may be determined in the motion analysis. A nonlinear skirt/or pile/soil interaction should normally be adopted for ULS analysis.

Risers may be represented by forces, as determined by motion analysis for free modes of behaviour or by static considerations for constrained modes of behaviour.

Ultimate strength criteria typically require calculation of the average stress level over an area, while fatigue criteria depend upon the local stress levels, e.g. in welded joints.

Linear elastic structural models are normally applicable for determination of response for ultimate and fatigue design checks. Non-linear structural effects may be accounted for in ALS checks as outlined in 10.6.

For space frame structures consisting of slender members a three-dimensional frame model may be used to calculate internal member forces and moments. The effect of joint eccentricity and flexibility, where significant, should be accounted for joint flexibility. Also, possible shear-lag and shear deformations should be accounted for. For space frames integrated with plated structures (e.g. deck) care should be exercised in modelling their interaction with beam elements.

For large volume thin-walled structures, three-dimensional FE membrane, plate or shell models should be used, possibly in combination with frame models.

Solid FEs may be required to be applied to represent stresses where three-dimensional stress conditions occur.

The structural response may often be considered as being divided into two broad categories as follows:

- global structural response, which requires global structural models that simulate, with sufficient accuracy, the effects of global actions on the structure;
- local structural response, which normally requires local structural models that simulate with sufficient accuracy the effect of local actions on the structure. Such local actions may be e.g. hydrostatic pressures, tank pressures, point forces, etc.

Hence, a hierarchical set of models may be used in analysis. First, a global model based on a relatively coarse FE shell model or a frame model may be used. Critical substructures may then be analysed by a three-dimensional FE model using appropriate boundary conditions and local actions. The effect of

secondary structural components (if not included in the space frame model) and local actions should be included in detailed local analyses.

Considerations of global action effects in local models may be undertaken using one of the methods listed as follows:

- mapping of actions (or responses) from global model to local model (sub-modelling), by e.g. using displacement or force boundary conditions obtained from the global analysis;
- integration of local model into global model;
- superimposing responses from the global model on the local model responses.

The extent of detail in a structural model is normally a balance between the need for accurate results with limited resources. Model extent, FE type, element size, and level of detail shall be consistent with the intended purpose of the structural model. The model extent should, however, be defined such that boundary conditions and actions can be imposed at well-defined, or well-understood, interfaces.

Boundary conditions should be defined which do not significantly affect the results of the analysis, e.g. artificially constrain support, or stiffen the structural model. Model boundaries should be located sufficiently far from the area of interest so that imposed boundary conditions do not significantly alter results in the area of interest.

Supplementary manual calculations for members subjected to local actions may be adequate in some cases, based on empirical formulas or basic engineering principles. The actions used for these calculations should come from the global FE analysis and from local actions acting on the structure.

Finite element analyses should be carried out with verified computer codes and FE models should be chosen with appropriate consideration of type of element, mesh size, and mesh shapes. Transition between different types of elements or sharp transitions in element size may distort the stress flow through a structural component, hence transitions should be placed away from the area of interest in structural analysis models. Mesh quality shall be reviewed to verify that distorted (and/or elongated) elements are not in areas of high stress concentration. Stresses should be interpreted with due consideration of the variation in discretization error (i.e. points with minimum uncertainty in stresses), stress extrapolation to boundaries and use of stress averaging techniques. The specific FE model (geometry, material properties, actions and boundary conditions) and the results should be verified.

10.3.3.2 Structural dynamic effects

Structural analysis may be performed quasistatically when dynamic effects (inertia and damping) due to structural vibrations are small.

The rigid body motions of a floating structure induced by waves are to be adequately accounted for by procedures specified in 10.2.

Relevant dynamic effects shall be considered in evaluation of the structural design. Dynamic response shall be considered when the period of steady-state action is close to some natural period of the structure or when the structure (or part thereof) is exposed to transient type of action. Dynamic effects may be important for example in connection with

- wave frequency actions if the natural (structural) period exceeds 2 s to 3 s,
- sum frequency effect of wave actions (e.g. springing, ringing),
- wave slamming, sloshing in tanks and other transient wave actions,
- wind action,
- earthquake action,
- mechanical impacts due to ships, icebergs or dropped objects,
- explosion action.

The dynamic model involves mass, damping and stiffness. Modelling of damping would normally have to be based on in-service experiences with similar types of dynamic structural systems. Some guidance is provided by Bartrop and Adams (1991). Caution needs to be exercised to avoid overestimation of damping when combining measured damping with damping implicit in theoretical models.

Dynamic response is obtained in the frequency or time domain. The linear response to a steady state action may conveniently be determined in the frequency domain. Transient response is most easily determined in the time domain.

If appropriately calibrated, the structural dynamic effects due to continuous wave action may be accounted for by a dynamic amplification factor in conjunction with the design wave approach, see 10.3.4.

Dynamic effects of impulse-type action may similarly be accounted for by using recognised charts or formulas for the dynamic amplification.

10.3.3.3 Stochastic effects

Wave, wind and seismic actions are stochastic of nature. In general procedures outlined in 10.2.4.2 and 10.2.4.3 can be used to determine action effects in the structure, foundation or mooring systems under stochastic actions.

10.3.3.4 Structural non-linear effects

Relevant considerations shall be given to non-linear action and response in the evaluation of the structural design.

Amplification of bending response ($P-\Delta$ effects) resulting from axial forces caused by rigid body motions or elastic deformations, should be accounted for.

Non-linear analyses may be based on engineering theories such as yield hinge or other theories of plastic mechanism, or FE models.

Non-linear FE analyses may be applied to determine the ultimate capacity of structural components, substructure or the total structure.

The method should include an appropriate model of all significant non-linear effects, including complete elasto-plastic behaviour, large strains and criteria for rupture. Geometrical imperfections and residual stresses should also be modelled, when they have a significant effect on the response. When using non-linear calculation models, adequate consideration shall be given to the fact that results depend on the action history. It shall be demonstrated that the least favourable action history is utilised. Generally it will be necessary to undertake parametric studies to evaluate different action histories in order to cover all modes of failure and structural elements.

The FE methods/computer codes applied to carry out nonlinear analyses, should be verified against test results or observed behaviour of full-scale structures, as well as known analytical solutions or other well-documented FE solutions.

The structure shall have sufficient ductility to develop the relevant failure mechanisms.

For structural parts that are subjected to cyclic actions, it shall be demonstrated that the structure can shake down, that repeated yielding does not lead to low cycle fatigue, incremental collapse or other failure modes. If linear elastic global analyses are carried out to check that the component strength is not exceeded, a demonstration that repetitive action does not cause failure, can be omitted.

Appropriate considerations shall be exercised in choosing FE type and mesh and ensuring that fabrication tolerances have been adequately accounted for in the analyses.

When applying nonlinear analyses as design basis it is important to take into account the engineering experience inherent in traditional design approaches. This implies that the ultimate capacity of components obtained by non-linear FE analyses should be consistent with that given in the relevant design code. This would normally require that the computer method to calculate ultimate strength of components subjected to compressive action be calibrated by adequate choice of theoretical model and geometrical imperfections.

10.3.4 Structural dynamic wave action analysis of bottom supported platforms

For fully constrained (bottom supported) platforms dynamic effects shall be accounted for when the natural period is above 3,0 and 2,0 s for determining steady wave action effects for ULS and FLS, respectively.

Such action effects may be determined by a static analysis if an adequate DAF is applied. This DAF is obtained by stochastic wave analysis as the ratio between the dynamic and quasistatic extreme action effect for ULS. For FLS the DAF is calculated as the ratio of the standard deviation of the dynamic and quasi-static action effects of axial force and moments in braces.

The dynamic amplification will vary over the structure. Horizontal accelerations of the topside and structural masses may be introduced to calibrate the quasi-static method of analysis. This simplified method could be used for a DAF up to 1,5. For larger DAFs a direct dynamic analysis should be applied.

10.3.5 Extreme high frequency response including, springing, ringing and whipping

Various types of higher order, interrelated high frequency wave actions may occur and cause extreme or repetitive action effects if the action frequencies coincide with natural frequencies of the structure, i.e. restrained modes of behaviour. Since the theoretical knowledge of these phenomena is limited, it is important to assess them jointly.

More specifically higher order actions that cause a transient action effect (ringing) may occur when steep, high waves encounter vertical components of structures with natural periods in the range from about 2 s up to 8 s.

Ringing effects is of significance only in combination with extreme first order wave frequency effects and ringing should be evaluated in the time domain with due consideration of first order wave action effects. The magnitude of the first response cycles is governed by the magnitude of force and its duration relative to the magnitude of the resonance period. The first ringing response cycles is not sensitive to the damping.

The ringing response is primarily of importance for ultimate and accidental limit state (in some cases for FLS) and SLS. Particular care is needed to establish the steep, unsymmetrical sea states that cause ringing.

Ringing action effects may be combined with low-frequency effects, by methods used for combining wave- and low frequency action effects.

Whipping is induced by bottom slamming or bow flare actions in ships and should be combined with the continuous wave action by properly accounting for the phase.

The significant uncertainties in current theoretical methods for predicting high frequency wave actions make it necessary to combine theoretical and experimental methods in order to determine characteristic action effects for design.

10.3.6 Stochastic dynamic wind action analysis

The effects of dynamic wind actions may be determined by a frequency domain analysis. One alternative is to use a modal formulation in this connection. The modal action effects may be combined with the SRSS method if the modes are not too close to each other. In case of modes having periods close to each other, the CQC method can be applied.

The extreme action effect due to wind can be determined by

$$S = S_s + \alpha \sigma_s \quad (17)$$

where

S_s is the static response due to the mean wind

σ_s is the standard deviation of the dynamic structural action effect

α is wind response peak factor

10.3.7 Seismic action effects

10.3.7.1 Action effects by response spectrum approach

The effect of earthquake actions for strength check of a soil-structure system may be obtained by a linear elastic model and use of modal superposition. An appropriate number of relevant modes to represent the action effect in question shall be included. Particular care is required to model local effects, i.e. subassemblies. The maximum of a given action effect in each mode can be obtained using the modal amplitude of the action effect and response spectrum in Figure 7, corresponding to the relevant modal period

, T , and damping value ζ . The damping in modes which do not involve soil deformation, or hysteric losses due to cracking or plasticity in the structure, should be carefully estimated because it may be less than the reference value of 5 %.

Deck, appurtenances, derricks, flare booms, critical piping and equipment need special consideration of global platform dynamics as well as possible local dynamic amplification.

An appropriate method should be used for combining modal responses for different directions. The CQC method may be used for combining modal responses in a given direction and the SRSS may be used for combining the directional responses. See e.g. Der Kiureghian (1980).

10.3.7.2 Global nonlinear strength analysis

If strength criteria are not fulfilled by action effects determined according to 10.3.7.1, it is to be demonstrated that the global structure-foundation system remains stable, without excessive deformations during the 10^{-4} earthquake, by taking into account system redundancy and force redistribution by inelastic deformations. A simplified approach based on a static nonlinear, or a dynamic nonlinear analysis may be used for this purpose.

In the simplified approach the action factor (of 1,3) for strength check is reduced depending on ductility and residual strength capability of the structure-foundation system demonstrated by a static nonlinear pushover analysis.

Direct nonlinear dynamic analysis should be carried out in the time domain, considering three standardised time histories, and using adequate models of the structure, foundation and soil as well as the surrounding water.

10.3.7.3 Other effects of earthquakes

Consideration should be given to the question whether earthquakes in the relevant area could have other effects, such as

- a) landslide,
- b) critical pore pressure build-up in soil,
- c) major soil deformations with subsequent deformations of foundation slabs, piles, skirts and pipes,
- d) low frequency waves in water,
- e) acoustic wave effects on submerged, non-flooded structural parts,
- f) tsunamis.

10.4 Extreme action effects for ultimate limit states

Most ultimate limit states occur when the structure has reached a state of nonlinear behaviour. However, the ULS check is normally performed by carrying out a linear elastic response analysis of the structure to determine forces or stresses in the individual components and by checking that the ultimate capacity is adequate component by component, using structural resistance formulations that incorporate non-linear effects occurring at component collapse.

Ultimate strength control typically considers average stress levels over an area which causes buckling. For the evaluation of buckling strength, mid-plate (membrane) element stress data should be used. For panels with large stress gradients, the variation of stress shall be considered in the buckling evaluation.

Component resistance is normally based upon experimental methods and generalised by parametric or by non-linear structural analyses, see 10.3.3.4. When multiple stress/force components affect the component strength, the strength may be expressed by interaction equations. The characteristic environmental action effect for ULS checks refers to a maximum, and in special cases, a minimum action effect corresponding to an annual exceedance probability of 10^{-2} . Various sets of combined response variables should be checked to ensure that the most critical set of characteristic response values is applied in the evaluation of the structure.

In non-linear structural analyses for ULS care should be exercised in application of partial resistance factors to cover uncertainties in the model as well as material and geometrical parameters.

Commonly used design methods are based upon the assumption that design values for action effect and resistance can be defined separately by introducing partial safety factors on the characteristic action effect

and resistance. In cases where integrated, non-linear analyses are used, care should be taken to ensure that equivalent levels of safety to those implicit in this NORSOK standard are obtained.

10.5 Repetitive action effects for fatigue limit states

Dynamic, principal stress ranges (or amplitudes) shall be used in the evaluation fatigue strength. The stress ranges should consider the variation and orientation of the principal stress vector components throughout the particular cycle being evaluated, e.g. wave. Extreme fiber stress (e.g. top/bottom surfaces in shell elements) shall be used based on the location of the material or weld being evaluated for fatigue performance.

The repetitive action for fatigue limit states is described by the distribution of stress ranges for welded structures. For basic (i.e. rolled and cast) material the joint distribution of mean stress and stress range is required. The stress may be expressed by a nominal hot spot or hot spot notch value. The latter stress includes the notch effect of weld geometry. It is crucial that the SN-curves applied are based on stresses defined in a consistent manner.

Stress concentration factors for fatigue analysis should be determined by appropriate detailed FE analysis, by physical models, by other rational methods of analysis, or by published formulas. If the relevant hot spot (notch) stress is obtained by FE analysis using solid or shell elements it may be necessary to determine the relevant stress by extrapolation procedure.

The main contribution to fatigue actions is normally from the local and global effect of waves and comes from moderate stress ranges. Fatigue design requires a description of the long-term variation of local stresses due to wave as well as possible sum-frequency wave actions, variable buoyancy, slamming, wave- or current-induced vortex shedding, or, mechanical vibration. The effect of local (e.g. pressure) and global actions shall be properly accounted for.

Account should be made of repetitive actions during fabrication, tow-out, installation as well as temporary and permanent, in-place conditions. For structures with oil storage, the repetitive effects of loading and unloading should be considered.

A linear elastic model of the structure is generally adequate. When significant, dynamic effects associated with structural vibration, shall be taken into account.

Miscellaneous hull appurtenances associated with risers, riser guides, anodes, mooring equipment should be evaluated for fatigue resistance, using local analyses. The calculation of actions on such components should be generated using water particle velocities and accelerations from diffraction analyses of the system that may possibly affect the mentioned components.

Stress ranges due to wide-band Gaussian or non-Gaussian response processes should be determined by an appropriate method of cycle counting, e.g. the rain-flow method. Simple conservative methods for combining (e.g. wave and high or low frequency responses) may be applied.

Screening in order to identify joints with high dynamic stresses and stress concentration, which require more detailed fatigue analyses, may be undertaken using the nominal member stress for the extreme event, an appropriate stress concentration factor and preliminary estimates of the Weibull stress range parameters describing the long-term stress distribution.

Detailed fatigue analyses should be performed using conservative deterministic analyses, spectral techniques, and in particular situations, by time domain analysis. Accurate analyses of local responses in the splash zone would require time domain analyses. Frequency or time domain stochastic approaches should be applied for dynamic sensitive structures.

10.6 Accidental damage limit state (ALS) analyses

The ALS design check requires evaluation of

- the structural damage caused by accidental actions,
- the ultimate capacity of structures with damage.

The large uncertainties associated with determining the accidental actions, normally justify the utilisation of simplified nonlinear analyses methods both to calculate the damage and the global ultimate strength of the

damaged structure. Such methods may be based on plastic mechanisms (yield hinge or line methods) with due recognition of possible premature rupture.

Non-linear FE analyses may also be applied on similar conditions as mentioned in 10.3.3.4.

Further details about such analyses may be found in the design standards for each type of material, e.g. steel, concrete, aluminium.

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