

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 Subject to development and publication of international standards, the relevant NORSOK standard will be withdrawn.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)

2.2 Informative references

Andersen and Løvseth (1992),	Andersen, O.J and Løvseth, J. (1992): "The Maritime Turbulent Wind Field. Measurements and Models" Final Report for Task 4 of the Statoil Joint Industry project, ALLFORSK, Norwegian University of Science and Technology
API RP 2N,	Planning, Designing and Constructing Structures and Pipelines for Arctic Conditions (1995)
Barltrop and Adams (1991),	Barltrop, N.D.P. and Adams, A.J. 1991: "Dynamics of Fixed Marine Structures", MTD/Butterworth-Heinemann.

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- 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 Preparednes Analysis (Rev. 2, 2001)
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- 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
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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).

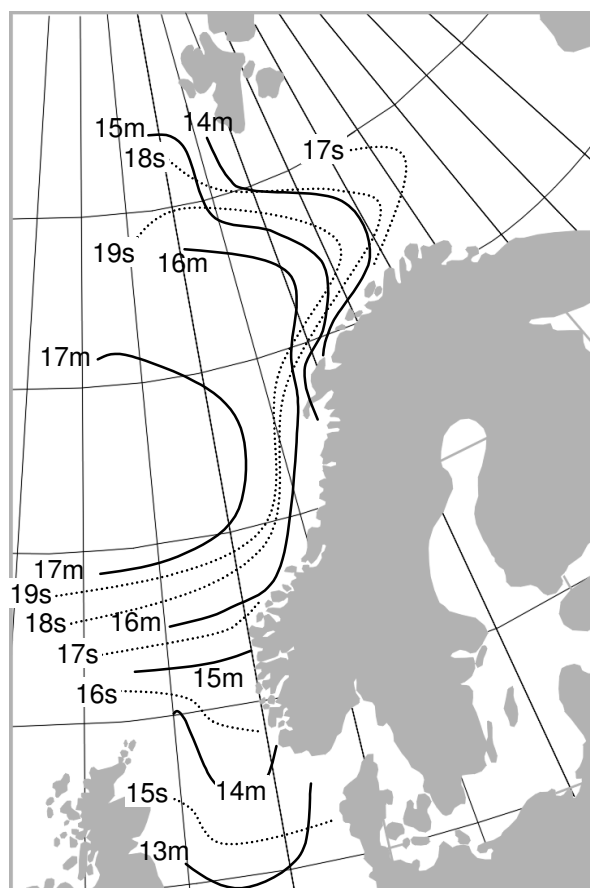


Figure 1 - Significant wave height H_s and related maximum peak period T_p with annual probability of exceedance of 10^{-2} for sea-states of 3 h duration. ISO-curves for wave heights are indicated with solid lines while wave period lines are dotted

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.

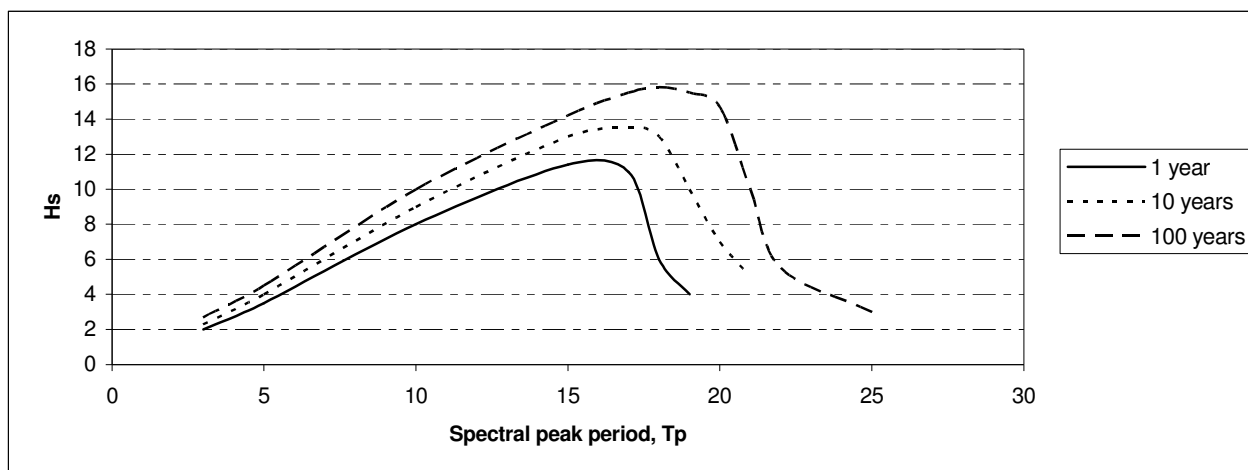


Figure 2 - Example contour line

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.

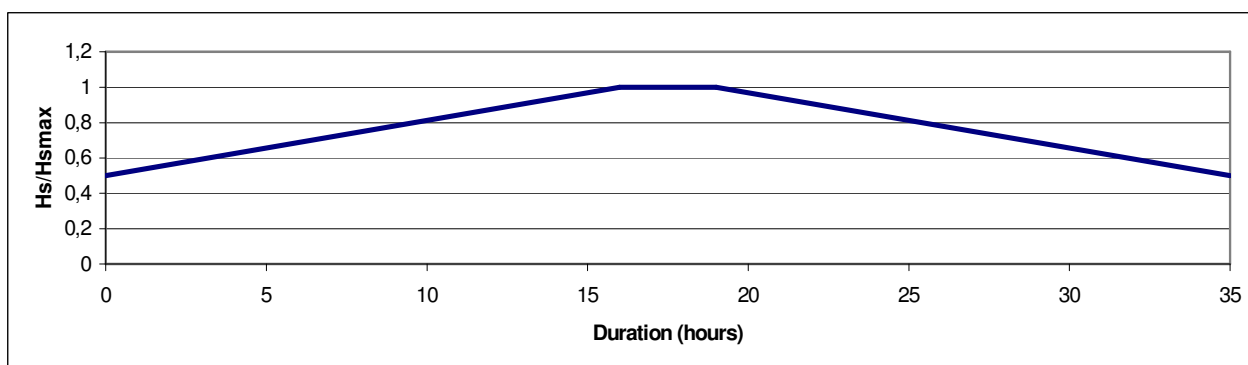


Figure 3 - Storm development for evaluation of the degradation of soil's resistance during cyclic action. The peak level is obtained for a sea-state with duration of 3 h

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.

Characteristic 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.