Guidelines relating to loads and load effects to regulations relating to loadbearing structures in the petroleum activities, issued by the Norwegian Directorate 7 February 1992. Last amended 20 January 1997.

CONTENTS

PREFACE

The purpose of the present guidelines is to show how the provisions relating to loads and load effects in regulations concerning loadbearing structures in the petroleum activities, can be met.

Functional requirements in the regulations signify that there are various ways in which to comply with the regulations. The Norwegian Petroleum Directorate's guidelines indicate one way to meet the requirements.

The guidelines alone are not legally binding. Consequently they do not prevent the selection of other technical and operational solutions, provided it can be documented that the selected solution meets the requirements stipulated in the regulations.

The guidelines are to be considered as a whole. The user should consequently exercise caution in using only parts of it.

The present guidelines replace guidelines for loads and load effects, issued by the Norwegian Petroleum Directorate 2 January 1987.

1
INTRODUCTION

The present guidelines offer guidance on how regulations concerning loadbearing structures in the petroleum activities can be met as regards the provisions of the regulations relating to loads and load effects. The guidelines are in particular related to the use of Sections 24-26 and Section 28 of the regulations. When reference is made in this text to provisions of the regulations, but without reference to a particular section, the abovesaid sections are relevant.

When reference in the present guidelines is made to «the regulations», it means a reference to the regulations concerning loadbearing structures in the petroleum activities.

2
PERMANENT LOADS

Permanent loads are loads that will not be removed during the time period comrised by the calculation, inter alia:
a) weight of the structure
b) weight of permanent ballast and equipment
c) external hydrostatic pressure from sea water up to the mean water level.

3
VARIABLE FUNCTIONAL LOADS

Variable functional loads are loads originating from normal operation of the structure, inter alia loads from:
a) persons
b) helicopters
c) cranes
d) stored liquids and goods
e) modules and structural parts that can be removed

f) boat impacts, fendering and mooring
g) weight of gas and liquid in process plants
h) pressure and temperature
i) variable ballast
j) installation and drilling operations
k) lifeboats

3.1 Crane loads - interrelationship aspects of different regulations
It follows from the regulations that crade pedestal and other attachment arrangments for lifting gear shall be in accordance with the provisions of the regulations. Account shall be taken to additional dynamic effects due to loading and discharging of ships.

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

3.2 Moorinig loads
According to Section 18 of the regulations, the operational limitations shall be documented by specifying where mooring may take place, under which weather conditions vessels are permitted to be moored, and the allowable size of the vessels permitted to be moored. Based on these restrictions the structure shall be designed for the most probable maximum load in this condition. Measures should be taken to avoid damage to the structure in the event of overloading. When maximum load transfer is calculated in a potentially weak link, a high characteristic value for the resistance of the link should be used, cf. Section 29 of the regulations. The calculation of mooring loads should take into account the fact that the mooring arrangement's load displacement characteristics may be changed during use (e.g. synthetic rope).

3.3 Deck loads
Deck loads for local design may be selected as:

NB

Wheel loads must be added where appropriate. Wheel loads man normally be distrubuted over an area of 0.3 · 0.3m. Point loads should in addition be applied to areas of 0.1 · 0.1 m in accordance with NS 3479, in the most unfavourable position, excluding wheel loads or distributed loads.

As recognised standard for loads on floors in accommodation and office sections, reference is made to ISO-2103-1986.

3.4 Ballasting
The structure should be designed to resist the maximum uneven distribution of ballast that may occur during fabrication, installation and operation. Loads that may occur during emptying of water or oil filled structural parts for condition monitoring, maintenance or repair shall be evaluated.

Tanks, pipes etc. shall be designed to resist the local pressures that may occur during normal operation.

3.5 Floors and railings

Walkways, stairs and landings should be designed for a local area load of 4 kN/m^2 . Temporary bars should be designed for 0.5 kN/m^2 . The load must then be placed horizontally on top of the supports.

4
ENVIRONMENTAL LOADS

4.1 General
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.

According the the regulations, the environmental loads shall be determined with stipulated probabilities of exceedance. The statistical analysis of measured data or simulated data should make use of different statistical methods to evaluate the sensitivity of the result. Methods based on an analysis of data above a given threshold shall be used with caution, since the results may depend on the chosen threshold. Correlation to the distributions should be tested by means of recognized methods. The Kolmogorov- Smirnov test may be used for statistically independent data.

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 particular caution should be exercised when determining design values. Model results can be used to extend measured time series, or to interpolate to places where measured data have not been collected. If the data are based on model results, a calibration shall be carried out against measured data which will provide results that are on the safe side.

4.2 Model tests and full-scale measurements
Characteristic load effects can be determined by model tests, based on observations of existing structures or by combining such tests and calculations. It is then a condition that the structure, the structural detail or the model examined has sufficient similarity with the case to be considered, and that tests or observations have been carried out which provide basis for reliable interpretation.

Model tests and full-scale measurements should be carried out in the case of:
a) new types of installations
b) structures with extensive motion
c) structures with major dynamic amplification

Wind tunnel tests should be carried out when wind loads are significant, and when:
a) another installation has a significant effect on the wind conditions applicable to the installation in question
b) there is a danger of aerodynamic instability
c) the wind loads are crucial to the stability of the structure.

NB

Figure 4.3.1 Significant wave height H_{mo} and related maximum peak period T_p with annual probability og exceedance of 10^{-2}. Isocurves for wave heights are indicated with solid lines. Wave period lines are dotted. A duration of 3 hours is presupposed.

When the wind conditions are determined for helicopter decks and structures with extensive motion, 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 recognized standard for wind tunnel tests.

Theoretical models for calculation of loads from icebergs or drift ice should be checked against model tests or fullscale measurements.

4.3 Hydrodynamic loads

4.3.1 Wave loads

If wave loads are of major importance in designing, the recording of data should have a duration of at least 10 years and should satisfy the requirements of the guidelines to the regulations concerning environmental data. If no measurements have been carried out in the area in question, a higher load coefficient should be selected for waves than the one used in the regulations, in order to deal with this uncertainty. Measured data may be replaced by model results as indicated in the guidelines Section 4.1.

4.3.1.1. General

Waves may be specified by:

a) a design wave, cf. subsection 4.3.1.2

b) a design sea state, cf. subsection 4.3.1.3

c) a long-term variation of the sea state, cf. subsection 4.3.1.4 and recognized wave theories.

4.3.1.2 Design wave

A design wave may be specified by the wave height H, the wave period T and direction. Different combinations of wave periods, wave heights and directions at the same probability level shall be considered in order to arrive at the most unfavourable values for the different load effects.

Maximum load effect is not always due to extreme wave heights, but also to waves of a defined length and extreme steepness. Examples are certain elements of floating installations with columns and pontoons.

The ratio between the wave height H for the largest wave and significant wave height H_{mo} may be set to 1.9 when the duration of H_{mo} is 3 hours.

The ratio between the period T for the largest wave and the peak period T_p in the wave spectrum may be selected equal to 0.92.

Significant wave height H_{mo} and peak period T_p are shown in Figure 4.3.1. These values may be used for exploration drilling and for early design phases. Interpolation can be used between the curves. In the detailed design, special studies of the relevant area should be carried out.

For crest evaluation calculations, the approach of Haring and Heideman (1978) can be used.

Wave theories as stated in Table 4.3.1 may be used when design wave is used. Other wave theories describing the wave kinematics may also be used. It is particularly important that the kinematics at still water level are well described.

At water depths shallower than the limits stated in Table 4.3.1, other wave theories shall be used, cf. Sarpkaya and Isaksen (1981).

NB
Table 4.3.1 Recommended wave theories

The possibility of freak waves should be considered.

The design wave method should be used with caution when determining load effects that are influenced by dynamic effects.

4.3.1.3 Design sea state
When using the design sea state method, significant wave height, peak period and direction shall be combined at the same probability level.

The JONSWAP spectrum may be used as extreme wave spectrum.

If the relevant load effect is sensitive with regard to the shape of the spectrum, alternative spectra should be considered.

When calculating the change in the soil's resistance during cyclic loads, 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 4.3.2 may be used.

Wave kinematics are usually determined by using linear wave theory. In cases where the load effect is influenced by dynamic effects or non-linear drag effects, linear theory should be modified to improve the description of the wave kinematics and loads in the splash zone.

When using the design sea state method, energy distribution around the dominating wave direction may be taken into account. In the case of sea states with significant wave heights less than 10 meters, the following spreading function may be used:

$$D\ (\Theta_m - \Theta) = C \cdot \cos^n(\Theta_m - \Theta)$$

where Θ_m is the mean wave direction, and where n is selected in the area 2-8 as the load is most unfavourable for each structure.

With regard to sea states with significant wave heights exceeding 10 meters, the energy distribution should be disregarded if it gives a reduced load effect.

4.3.1.4 Long-term variation of waves
The variation of waves over a long period can be described as a number of stationary sea states with associated probabilities of occurrence. Alternatively, the long-term variation of

waves can be described by means of individual waves, where the long-term variation of waves is represented by a number of wave height groups characterized by a wave height, a wave period and a number of waves in the group. This method is not recommended if the dynamic effects are very significant.

The methods using long-term variation of waves may be applied to establish a probability distribution for the load in question. The probability distribution may be used to establish both extreme load and cumulative load.

The selection of short term states or wave height groups should be made in such a way that the long term variation of the wave loads is described with sufficient accuracy with regard to variation both of wave height and of period. In this connection it is important to have good knowledge of the structure's sensitivity to variations both in wave height and in period.

NB
Figure 4.3.2 Storm development for evaluation of the breakdown of soil resistance during cyclic loads. The peak level has a duration of 6 hours.

4.3.2 Current velocities
If current velocity is of great significance to the design, current velocity measurements should be carried out at the location in question. The recordings should have a duration of at least 1 year. If no measurements have been carried out or there has been measured for a duration less than one year at the location in question, a higher load coefficient for current should be selected than that mentioned in the regulations. Model results may replace or be used in addition to measured data, cf. subsection 4.1.

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 4.3.3.

If no exact measured data are available, the wind induced current velocities at still water level may be selected equal to 2% of the wind velocity, cf. subsection 4.3.1.

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 Det norske Veritas (1989).

In addition, the effects of other currents shall be considered in each separate case, including the effects of:
a) coast and ocean currents
b) local eddy currents
c) currents over steep slopes
d) currents caused by storm surge
e) internal waves

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

4.3.3. Calculation of hydrodynamic loads
Current and wave loads shall be considered in conjunction, as the particle velocity is found by vectorial addition of the current and wave particle velocities.

NB
Figure 4.3.3 Maximum tidal current in m/s at vernal equinox spring tide.

4.3.3.1 Slender tubular structural elements
For structures with small motions, the wave loads can be calculated as follows:
a) If the Keulegan-Carpenter figure (KC) is less than 2 for a structural element, the loads may be found by means of potential theory:
aa) If the ratio between the wave length L and the tubular diameter D is greater than 5, the inertia term in the Morrison formula can be used with $C_M = 2.0$.
ab) 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 load can be calculated by means of the Morrison formula, with C_D and C_M given as functions of the Reynold figure Re, the Keulegan-Carpenter figure KC and relative roughness. With regard to frame works constructions, C_D and C_M may be chosen from DnV(1989) part 3 ch 1 part 5, point C201-C208.

4.3.3.2 Large volume structures
For large volume structures, the loads should be calculated on the basis of diffraction theory. Surface elevation caused by the structure shall be taken into consideration.

4.3.3.3 Structures with significant motion
When the ratio between motion amplitude and diameter is greater than 0.2, the relative motion between the structure and water should be taken into consideration when calculating wave and current loads by means of the Morrison formula. The dynamic equilibrium equation may then be formulated as follows:

$$m\ddot{X} + c\dot{X} + kX = C_{d\rho} R\left|u - \dot{X}\right|(u - \dot{X}) + C_{m\rho}\pi R^2(\dot{u} - \ddot{X}) + C_{f\cdot\rho}\pi R^2\dot{u}$$

The load coefficients may be added to the load effect in such calculations.

The following parameters are used in the equation: m, c and k are mass, damping and rigidity, respectively.

C_d , C_m og C_f are hydrodynamic coefficients

Reference is made to Sarpkaya and Isachsen (1981)

X, $\dot{X}$ og $\ddot{X}$ are the installation’s displacement, velocity and acceleration

u and $\dot{u}$ are the velocity and acceleration of the liquid particle.

The formulation with relative velocity in the equation is uncertain for installations with little 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.

Strip theory may be used to determine loads and motion for oblong, floating structures.

Average drift load in waves may be determined by calculations based on diffraction theory.

Slowly varying drift loads and load effects should be based on calculations, model tests or full-scale measurements, cf. Det norske Veritas (1990). Possible dynamic amplifications shall be taken into account. Herfjord and Nielsen (1991) indicate uncertainties linked with the various calculation methods.

Springing and ringing shall be considered.

Higher order sum frequency loads and load effects such as springing and ringing shall be taken into consideration.

Current induced vibrations shall be taken into account, e.g.:

a) vortex shedding

b) Mathieu instability

c) instability caused by varying orientation of the structure in relation to the wave and current direction (galloping)

d) back wash effects from mobile units in the vicinity.

Loads caused by vortex shedding may be of significant importance in the design of slender structures with small damping. With regard to wind as well as current and waves, the question whether extensive parts of the structure may be subjected to vibrations caused by vortex shedding should also be considered.

Tuning of the vortex shedding frequencies and the natural vibration frequencies of the structure may cause resonant vortex shedding in a relatively wide range of velocities. This shall be taken into account in the fatigue calculations. The effects of resonant loads during construction and transportation shall be included in the calculations.

4.3.3.4 Wave slamming

Wave slamming and shock pressure may be calculated according to Det norske Veritas Classification Note 30.5 or Ridley (1982).

4.3.3.5 Abnormal wave loads

The wave load with annual probability of exceedance of 10^{-4} can be assumed to be equal to 1.2 times the wave height with annual probability of exceedance of 10^{-2}.

4.4 Wind loads

If the wind loads are of great significance to the design, the recordings should have a duration of at least 10 years and should satisfy the guidelines to the regulations concerning environmental data. If no measurements have been carried out in the area in question, a higher load coefficient should be selected for waves than the one used in the regulations. Measured data may be replaced by model results as indicated in the guidelines Section 4.1.

4.4.1 Description of wind

For a short term condition the wind may be described by means of an average wind velocity overlain by a fluctuating wind gust with a mean value equal to zero.

The average wind velocity at 10 meter above sea level the characteristic value with an annual probability of excedance of 10^{-2} can be chosed as 41 m/s (10 min average) or 37 m/s (1 hour average) for the whole continental shelf. The characteristic value with an annual probability of excedance of 10^{-4} can be chosed as 48 m/s (10 min average) or 42.5 m/s (1 hour average),

The wind velocity as a function of height and mean period may be calculated as follows:

$U(z,t) = U(z_r, t_r)\,(1.0 + 0.137 \ln(z/z_r) - 0.047 \ln(t/t_r))$

where z_r= 10 m and t_r= 600 s

or, in the case of extreme wind conditions, the characteristic wind velocity $u(z,t)(ms^{-1})$ at a height z (m) above sea level and corresponding mean period t less than or equal to t_o = 3600 s may be calculated as

$u(z,t) = U(z)\cdot(1-0.41\cdot I_u(z) \ln(t/t_o))$

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

$$U(z) = U_\circ \cdot \left[1 + C \cdot \ln\left(\frac{z}{10}\right)\right]$$

$$C = 5.73\cdot 10^{-2} \cdot \left(1 + 0.15 \cdot U_\circ\right)^{1/2}$$

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

$$I_u(z) = 0.06 \cdot \left[1 + 0.043 \cdot U_\circ\right] \cdot \left(\frac{z}{10}\right)^{-0.22}$$

where $U_\circ(ms^{-1})$ is the 1 hour mean wind speed at 10 m

For structures and structural elements for which the wind fluctuations are of importance the following 1 point wind spectrum shall be used for the longitudinal wind speed fluctuations

$$S(f) = \frac{320 \cdot \left(\frac{U_\circ}{10}\right)^2 \cdot \left(\frac{z}{10}\right)^{0.45}}{\left(1 + \tilde{f}^{\,n}\right)^{\frac{5}{3n}}}$$

$$\tilde{f} = 172 \cdot f \cdot \left(\frac{z}{10}\right)^{2/3} \cdot \left(\frac{U_\circ}{10}\right)^{-0.75}$$

where n = 0.468 and where
- S(f) ($m^2 s^{-2}$/Hz) is the spectral density at frequency f (Hz)
- z (m) is the height above sea level
- $U_\circ \left(ms^{-1} \right)$ is the 1 hour mean wind speed at10 m above sea level.

The correlation coefficient between the spectral densities of the longitudinal wind speed fluctuations of frequency f between 2 points in space is given by the square root of the 2 points coherence spectrum.

The recommended coherence spectrum between 2 points

- at levels z_1 and z_2
- with across-wind positions y_1 and y_2
- with along-wind positions x_1 og x_2

is given by

$$Coh(f) = \exp \left\{ -\frac{1}{U_\circ} \cdot \left[\sum_{i=1}^{3} A_i^2 \right]^{\frac{1}{2}} \right\}$$

where

$$A_i = \alpha_i \cdot f^{r} \cdot \Delta_i^{q} \cdot z_R^{P}$$

$$z_R = \frac{\left(z_1 \cdot z_2 \right)^{\frac{1}{2}}}{10}$$

and where coefficients α, p, q, r and the separation Δ are given in Table 1.

NB
Tabell 1. Coefficients and separation for the 3-D (i =1.2.3) coherencespectrum. Note that separations are given by absolute values.

4.4.2 Static wind loads
Structures or structural elements that are not sensitive to wind gusts, may be calculated by regarding the wind load as a static load. In the case of structures or structural parts where the maximum length is less than 50 m, 3 seconds wind gusts may be used when calculating static wind loads.

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. For structures or structural elements that are exposed to simultaneous wind and wave loads, and where the wave load is the dominating load, 1 minute of mean wind may be used in combination with extreme wave loads.

Unless more accurate calculations are performed, the wind loads may be calculated according to NS 3479 or Det norske Veritas Classification Note 30.5.

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

$C_D = 0.7$ for $Re > 5\ 10^5$

$C_D = 1.2$ for $Re < 5\ 10^5$

In the case of tubular structures covered with ice, $C_D = 1.2$ should be used for all Reynolds figures.

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

With regard to structures with a natural period less than 10 seconds, a 10 min. mean period for average wind velocity may be selected.

For structures with a natural period exceeding 10 seconds, a 1 hour mean period for average wind velocity may be selected.

For structures of great extension, the spacial variation of the fluctuating wind should be taken into account.

Possible torsion effects should be particularly 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.

Consideration of loads from vortex shedding on individual elements due to wind may otherwise be based on DIN-4133 Anhang A or Eurocode 1. Guidelines on how these standards may be applied offshore, are provided by Oppen (1995). Vortex induced vibrations of frames should be considered in addition.

NB
Tabell 4.5.1 Ice loads with annual probability of exceedance 10^{-2}

4.5 Snow and ice loads
4.5.1 Snow loads
The snow loads given in NS 3479 for the individual municipalities may be used as extreme snow load close to the shore. For other areas where more accurate meteorological observations have not been performed, characteristic snow load may be set equal to 0.5 kPa for the entire Norwegian continental shelf. The shape factors given in NS 3479 may be used.

4.5.2 Ice loads
4.5.2.1 Accumulated ice
Values for thickness of accumulated ice caused by sea spray or precipitation may be selected as indicated in Table 4.5.1. These may be regarded as two independent load cases.

When calculating wave, current and wind loads, increases in dimensions and changes in the shape and surface roughness of the structure as a result of accumulated ice may be included by assuming that:
a) ice from sea spray 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 stabilized structures.

NB
Figure 4.5.1 Ice limits in the Barents Sea with annual probability of exceedance of 10^{-2} (solid line) and 10^{-4} (dotted line).

4.5.2.2 Frost burst
The effects of ballast water, fire water etc. which may freeze shall be taken into account.

4.5.2.3 Sea ice and icebergs
Loads from sea ice and icebergs shall be taken into account when structures are located in areas near shore, in Skagerrak, 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:
a) the possibility of icebergs and sea ice
b) type of sea ice (first year ice; ice several years old) and characteristic features (ridges, large interconnected floes and individual floes, distribution etc.)
c) sea ice thickness
d) size and shape of icebergs
e) velocity and direction of drifting sea ice and icebergs
f) mechanical properties of the ice.

NB
Figure 4.5.2 Limits for collision with icebergs with a probability of exceedance og 10^{-2} (dotted line) and 10^{-4} (solid line).

In connection with exploration drilling and in the early phases of a development in the Barents Sea, monitoring of sea ice and icebergs shall be considered. There must 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 unacceptabe, 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 4.5.1. 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 10-20 % 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 loads 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. American Petroleum Institute (1988) Chapters 3 and 4 may then be used.

The probability of collision between an iceberg and an installation with annual probability of exceedance of 10^{-2} and 10^{-4} in the Barents Sea is shown in Figure 4.5.2. Icebergs in considerable numbers have been observed on the coast off East Finnmark in 1881 and in 1929.

When designing against icebergs the same principles as stipulated in subsection 6.3.2 of the guidelines may be used.

4.6 Earthquake loads

4.6.1 Design earthquake loads

The ISO standard ISO-3010-1988 may be regarded as recognized standard for design against earthquake loads.

Earthquake loads should be determined on the basis of the relevant tectonic conditions, historical seismological data and measured time histories for earthquakes in the relevant area. When determining earthquake loads on the structure, interaction between the soil and the structure should be taken into consideration. The earthquake motions at the location may be described by means of spectra or time histories.

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 loads from the time history analyses may be taken as basis for the load calculations. The time series shall be selected in such way that they are representative of earthquakes on the Norwegian continental shelf at the given probability of exceedance.

Normally earthquakes with an annual probability of exceedance of 10^{-2} can be disregarded.

4.6.2 Earthquake loads

Unless more accurate examinations are performed, spectra may be used for bedrock (normalized to 10 m/s^2 acceleration at 40 Hz for the horizontal main component) as illustrated in Figure 4.6.1. The spectra may be used together with the accelerations given in figures 4.6.2 and 4.6.3. If for instance the acceleration is 2.5 m/s^2, the spectrum with an annual probability of exceedance of 10^{-4} is obtained by multiplying the normalized spectrum in Figure 4.5.1 with 0.25. The spectrum is based on the total structural damping (including energy loss to the soil) being 5 percent of critical damping. The spectrum must be adjusted if the damping is different 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. For scaling of damping between 2 and
10 % the following formula may be used:

$$D = -\ln\left(\zeta / 100\right) / \ln\left(20\right)$$

where ζ is damping.

The given spectra are calculated for bedrock. For unfixed masses the spectral values for each individual period in Figure 4.6.1 are multiplied by a factor k that is dependent on whether the unfixed masses are soft or stiff (hard). The factor k is determined according to the formula:

$$k = 1/3 * k_{min} + 2/3 * k_{max}$$

where k_{min} is the lowest spectral condition for a given period in the hatched areas in Figure 4.6.4, and k_{max} is the highest spectral condition.

The figures are based on Bungum and Selnes (1988).

NB
Figure 4.6.1 Response spectra (normalized to 10 m/s^2 acceleration at 40 Hz) with 5 % damping. Maximum accelertion, velocity and displacement for the curves a) and b) are as follows:

Curve a (earthquake with an annual probability of 10^{-4}) 30 m/s^2 , 0.72 m/s, 0.18 m

Curve b (earthquake with an annual probability 10^{-2}): 30 m/s^2 , 0.60 m/s, 0.14 m

Spectra may only be used for bedrock, and shall be adjusted for the relevant soil conditions by the factors in Figure 4.5.4.

NB
Figure 4.6.2 Map of seismic zones with an annual probability of exceedance of 10^{-2} . The hatched areas indicate the largest acceleration level for bedrock with 5 % damping (Bungum and Selnes, 1988).

NB
Figure 4.6.3 Map of seismic zones with an annual probability of exceedance of 10^{-4} . The hatched areas indicate the largest acceleration level for bedrock with 5 % damping (Bungum and Selnes, 1988).

NB
Figure 4.6.4 Ratio between spectral values for relevant soil conditions and bedrock for earthquakes with an annual probability of exceedance of 10^{-2} (top) and of 10^{-4} (bottom). The hatched areas indicate the variation area for calculated values, the unbroken lines show the reinforcements for analysed soil conditions. (Bungum and Selnes, 1988).

4.6.3 Calculation of load effect
The earthquake motion can be described by two ortogonally horizontal vibrations and one vertical vibration acting simultaneously, but which are assumed to be statistically independent. One of the horizontal vibrations should be parallel to a main structural axis. Unless more accurate calculations are performed, the ortogonally horizontal component may be set equal to 2/3 of the main component and the vertical component equal to 2/3 of the main component, referred to bedrock. The earthquake may be scaled with the given factors.

When the response spectrum method is used, the number of vibration modes included should at least be sufficient to ensure that at least 90 % of the total energy is taken into account. This percentage requirement shall be met in respect of earthquake motions of deck structures and main loadbearing structures. If the modal super position principle is used, at least 90 % of the modal mass shall be included. This means that the sum of generalized mass from the vibration modes that are included, shall be 90 % of the total mass.

For earthquake loads with an annual probability of exceedance of 10^{-4} , it shall be documented that the capacity of the structure and the soil is sufficient to absorb the resulting energy, so that progressive collapse is avoided.

Furthermore, consideration should be given to the question whether earthquakes in the area in question 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.

4.7 Other environmental loads
4.7.1 Marine growth
4.7.1.1 General
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 loads, increased weight, increased hydrodynamic additional mass and may influence hydrodynamic instability as a result of vortex shedding and possible corrosion effects.

4.7.1.2 Thickness of marine growth
In the calculation of structural loads, 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 4.7.1 may be assumed.

NB
Table 4.7.1 Thickness of marine growth. The water depth refers to mean water level.

Water depth 56-59° N 59-72° N

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 formula.

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

4.7.1.4 Cleaning
Should marine growth exceed the values for which the installation is documented, cf. Section 22, cleaning may be omitted if a new analysis shows that the structure has sufficient strength.

4.7.2 Water level, settlements, subsidence and erosion
When determining water level in the load calculations, the tidal water and storm surge shall inter alia be taken into account. 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 shall be considered. Reservoir settlements and subsequent subsidence of the seabed should be calculated as a conservatively estimated mean value.

As a precautionary measure, a steadily increasing water level with up to 5 mm per year as a result of the greenhouse effect, should be taken into account.

The tidal water constants M2, S2 and K1 may be taken from Gjevik, Nøst and Straume (1990).

4.7.3 Splash zone
The splash zone in Section 32 of the regulations may be calculated from 4 m below the lowest tide to 5 m above the highest tide. Cf. also the discussion in the guidelines to the regulations, Section 32.

NB
Table 4.8.1 Combination of environmental loads with expected mean values and annual probability of exceedance 10^{-2} and 10^{-4} .

4.8 Combinations of environmental loads
For environmental loads with an annual probability of exceedance of 10^{-2} and 10^{-4} , the load combinations may be selected in accordance with Table 4.8.1.

Unless a more accurate method is used, it shall be assumed that the most unfavourable wind, current and wave loads occur in the same storm. Within the storm, wind and waves can be taken to be uncorrelated.

5
DEFORMATION LOADS

Deformation loads are loads caused by inflicted deformations, also when they are a result of the structure’s function or the surrounding environmental conditions.

5.1 Temperature loads
Loadbearing structures shall be designed for the most extreme temperature differences they may be exposed to. This applies, inter alia, 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 Figures 5.1, 5.2 and 5.3. 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.

Temperature changes as a result inter alia of the greenhouse effect should be taken into account. A steadily increasing air temperature of up to 0.02 °C per year in respect of the highest temperature and up to 0.03 °C in respect of the lowest temperature may be assumed.

5.2 Loads due to fabrication

During design, reasonable tolerances shall be assumed, and account shall be taken for possible loads that may arise from restraints.

The effects of errors, e.g. geometric deviations or defects exceeding the tolerance limits, should be considered by the person responsible for the design.

5.3 Loads due to settlement of foundations

Effects of uneven settlements of the foundation shall be considered. Loads on the structure from risers and drillstring as a result of foundation settlements should be considered. Local reaction loads on the structure during installation due to uneven seabed or boulders shall be considered, cf. Section 23 and Section 24 of the regulations.

NB

Figure 5.1 Highest and lowest air temperature with an annual probability of exceedance of 10^{-2} . The temperatures are given in degrees Celsius.

NB

5.2 Highest surface temperature in the sea with an annual probability of exceedance of 10^{-2} . The temperatures are given in degrees Celsius.

NB

Figure 5.3 Lowest surface temperature in the sea with an annual probability of exceedance of 10^{-2} . The temperatures are given in degrees Celsius.

6
ACCIDENTAL LOADS

6.1 General

Accidental loads shall be understood as loads to which the structure may be subjected in connection with incorrect operation or technical failure. When accidental loads are determined, factors that may be relevant shall be taken into account. Such factors may be personnel qualifications, operational procedures, the arrangement of the installation, equipment, safety systems and control procedures.

Relevant accidental loads and their magnitude should be determined on the basis of a risk analysis. With regard to the planning, implementation, use and updating of such analyses, reference is made to the risk analyses regulations.

6.2 Fire and explosions

The following types of fire should inter alia be considered:
a) burning blowouts
b) fire from leaks in risers or process equipment
c) burning oil on the sea
d) fire in equipment or electrical installations
e) fire on helicopter deck
f) fire in living quarters

The effects of explosions should be considered, such as:
a) ignited gas clouds
b) explosions in enclosed spaces.

Structural designs should be selected so as to limit the effect of fire and explosion.

The fire load intensity may be described as a function of time and place. The explosion load may be described by the pressure development.

According to Section 20 of the regulations, changes in material properties shall be taken into account when the changes are caused by fire and explosion. Fire and explosion should be assumed to occur at the same time.

In the case of hydrocarbon fires, the temperature-time curve should be used as basis for demonstrating adequate fire resistance, cf. regulations relating to explosion and fire protection.

Extinguishing fires may lead to use of large quantities of water. The consequences of such additional loads should be taken into account.

6.3 Impact loads

6.3.1 General

Impact loads are characterized by kinetic energy, impact geometry and the relationship betweed load and indentation.

Impact loads may for instance be caused by:
a) vessels in service to and from the installation;
b) tankers loading at the field;
c) ships and fishing vessels passing the installation;
d) floating installations, such as flotels;
e) aircraft on service to and from the field;
f) falling or sliding objects;
g) fishing gear;

h) icebergs or ice.

6.3.2 Collisions with vessels
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. Design values for collision are determined based on an overall evaluation of possible events.

Experience has shown that the design should take into account collision with 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.

During exploration drilling and in the early phases of designing, the mass of supply ships should normally not be selected less than 5000 tons and the speed not less than 2 m/s. A hydrodynamic additional mass of 40 % for sideways and 10 % for bow and stern impact can be assumed. Further information on calculation of collisions has been prepared by Det norske Veritas (1981).

The most probable impact points and the most probable impact geometry should be used as basis for the analysis. If a central impact (impact load through the vessel's centre of gravity) is physically possible, this impact situation should be analysed.

The behaviour of the vessel and the installation during the impact, and thus the distribution of impact energy between kinetic rotation and translation energy and deformation energy, can be determined by dynamic equilibrium or energy considerations.

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. In addition, the global effect of collision loads shall be considered, cf. Section 20.

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 loads on the other installation.

6.3.3 Falling objects
Loads should be considered inter alia based on the following types of incidents:
a) alling cargo from lifting gear
b) falling lifting gear
c) unintentionally swinging objects
d) loss of valves designed to prevent blowout 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, loads from falling objects on to the deck may be based on the safe working load for the lifting equipment. This load should be assumed to be falling from the lifting gear from the highest specified lifting height and at the most unfavourable place.

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 loads should also be considered.

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

Norsok standard U-CR-001 provides values for accidental loads on subsea installations. These values may be used in evaluations during an early phase.

6.4 Change in intended pressure difference or buoyancy

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, shall be considered, cf. Section 16.

For a structure in floating condition, the question whether a changed floating condition e.g. as a result of unintentional flooding of compartments, tubular structural elements and tanks, will lead to unacceptable safety for people, the environment or assets and financial interests, shall be assessed. For floating structures it should be considered whether a subsea gas blowout can cause loss of buoyancy or motions of significance to stability, to the anchoring system or to the distance to other installations.

6.5 Helicopter loads

If specific information about the weight distribution of the undercarriage of a helicopter is not available, the helicopter deck should be designed on the preassumption that any point on the deck may be subjected to a single load equal to 75 % of the total weight of the heaviest helicopter used. The single load can be assumed evenly distributed over an area of 0.093 m^2. Reference is made to regulations relating to commercial aviation to and from helicopter decks on fixed and mobile offshore installations, issued by the Civil Aviation Administration 28 December 1992, including also sub-section 1.1, last paragraph (scope).

The loadbearing structures below the helicopter deck should be designed for a load 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.

Helicopter decks that meet the requirements of Det norske Veritas' class for helicopter decks will also satisfy the requirements according to the present regulations.

7. LOAD EFFECT ANALYSES

7.1 General

By load effect analyses is meant all calculations used to determine the load effects. The term thus comprises both movement analyses, finite element analyses, static as well as dynamic, non-linear element analyses and other types of analyses which analyse by means of element methods and similar.

Load effect analyses must always be assessed in connection with the nature and effect of the loads and the subsequent design or capacity control. The following paragraphs are consequently only applicable for analyses linked with the assumed loads and design.

7.1.1 Requirements to analysts
The person professionally responsible for the implementation of load effect analyses shall be familiar with the principles on which the analysis is based, with all its pre-assumptions and with the limitations of the methods used.

7.1.2 Requirements to analysis tools
The analysis tools shall be tested and suitable for the task. A suitable user's description with necessary guidelines and information on limitations connected with the method used must be available. If an analysis programme is used for new types of structures or in a way where there is no previous experience in the use of the programme, it should be checked that the necessary accuracy is achieved. This can be done by checking results and by model tests, by another method or another programme or by test analysis of a simplified problem.

7.1.3 Requirements to calculation model
Any calculation model is an idealisation of the structure being analysed and it must always be ensured that the chosen model represents the structure and the loads in a way that makes the method that is used achieve results with adequate accuracy or with conservative results. An assessment must be carried out to the effect that the chosen model represents the structure in a way which corresponds to the calculation model assumptions and that the chosen simplifications are compatible with the subsequent design.

7.1.4 Requirements to the analysis
One's own checking of the analysis results should comprise checking that:
- the element type is suitable to the purpose
- boundary conditions function as assumed
- the deformation process and size is as expected
- calculation of structure parts (local, detailed analyses) has adequate by applied loads and boundary conditions for all load situations. Test analyses may be used
- the main mode of application seems probable in comparison with other methods or results from other corresponding analyses.

7.2 Movement analyses
7.2.1 General
Linear wave theory can to a large extent be used for calculation of wave induced movements for ships, semi-submersible platforms, tension leg platforms and other floating structures. Linear superposition can then be used to calculate load effects in irregular waves.

Time series analyses may be necessary to determine load effects from extreme or breaking waves, structures with non-linear variation of the water plane area, non-linear soil foundation behaviour, non-linear behaviour by the anchoring system or similar. When analysing movements for structures with low damping, such as rolling movements for ships, it is necessary to include viscous damping in order to obtain realistic results. For structures with natural periods in excess of 25 seconds, slow by varying effects from waves and wind must be included. For structures with natural periods between 3 and 8 seconds, second order effects such as ringing and springing must be taken into account.

With regard to structures which have been optimized with respect to minimal linear wave loads, experience has shown that second order effects will often be relevant. Movement analyses for new types of structures where the results cannot be checked against previous experience, should be checked against model tests.

7.2.2 Anchoring analyses
Anchoring analyses of catenary moored units can be carried out according to the DNV «Rules for Classification of Mobile Offshore Units, Part 6, Chapter 1» (Det Norske Veritas 1989) or as per APE «Recommended Practice 2FP1 Recommended Practice for Design, Analysis, and Maintenance of Moorings for Floating Production Systems» (1993). Guidelines on analysis of tension legs can be found in API RP 2T(1987).

7.2.3 Time series analyses
Guidelines for implementation of time series analyses can be found in Chapter 7 of Technical and Research Bulletin 5-5 «Guideline for site specific assessment of Mobile Jack-up units». (Society of Naval Architects and Marine Engineers 1991).

7.3 Ultimate limit state analyses
7.3.1 Linear analyses
Most ultimate limit states occur only when the structure has reached the state of non-linear behaviour. In most cases, however, it will prove to be acceptable to carry out linear analyses and check capacity part by part. This is the most common ultimate limit state analysis method. The basis for this is that it is a fundamental requirement to structures that they display a ductile behaviour. Ductility means that a structural detail can be deformed in excess of the deformation at characteristic resistance without losing its loadbearing properties. The ductility of the structure will ensure that the the structure’s residual stresses, e.g. from temperature, differential settlements fabrication and similar, in most cases can be disregarded in the analysis.

In the case of structures with repetitive loads, it must be checked that the redistribution which is presumed, does not lead to collapse due to repeated yielding. If linear elastic analyses are carried out checking that the yield stress is not exceeded, check for repeated yielding is, however, not required. In planning and assessing the results from finite element analyses, the accuracy achieved from the use of the element type for the relevant application, must be assessed. In calculating the load effect from the design wave method, it must be checked that the waves selected produce conservative results for all modes of failure and structural parts. In calculating the load effect on the basis of a designing sea condition, the linearization of the wave forces must be made in a way that does not underestimate the load effects.

7.3.2 Non-linear element analyses
The use of non-linear element analyses requires more checking of results and higher competence for those carrying out the analyses than the use of linear analyses. It is of particular importance that the engineering experience which is inherent in design methods based on traditional (linear) analyses, is taken into account when non-linear analyses are used as design basis.

The calculation programme used must be suitable to the task in order to ensure that no critical modes of failure are overlooked. With regard to structural parts subjected to cyclic loads, it must be checked that repeated yielding does not lead to failure (low cycle fatigue). Special control measures must be initiated when non-linear analyses have not been checked against tests or behaviour of full-scale structures. Such control measures may be checking against other element programmes, testing of the programme on simple, verifiable problems, comparison with results achieved with linear programmes or similar. When using non-linear calculation models, the fact that results are dependent on the load history, must be taken into account, and it must be shown that the least favourable load history has been used. Generally it will be necessary to check different load histories in order to cover all modes of failure and structural elements.

7.4 Fatigue analyses

7.4.1 Global analyses

In the case of fatigue analyses it is necessary to represent the structure’s actual rigidity with adequate accuracy. Stiffness contributions from non-modelled structural elements or equipment must be included if they are significant. In cases where the stiffness is uncertain, e.g. soil stiffness, it may be necessary to vary the stiffness parameters in order that all parts of the structure that is analysed, get results that represent the load effects from expected load history with adequate accuracy. Deterministic methods must be used with caution in respect of dynamically sensitive structures.

7.4.2 Local analyses

When fatigue analyses are carried out for particular structural parts by analysing only part of the structure, it must be checked that applied loads and boundary conditions give a satisfactory representation of the stress distribution for all load situations.

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